GOVERNMENT OF INDIA
MINISTRY OF HOUSING AND URBAN AFFAIRS

MANUAL ON WATER SUPPLY AND TREATMENT SYSTEMS (DRINK FROM TAP)

PART A: ENGINEERING - PLANNING, DESIGN AND IMPLEMENTATION

FOURTH EDITION - REVISED AND UPDATED

CENTRAL PUBLIC HEALTH AND ENVIRONMENTAL ENGINEERING ORGANISATION

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In Collaboration with

MARCH 2024

In keeping with the advancements in the sector, updates as and when found necessary will be hosted on the Ministry's website: http://mohua.gov.in and CPHEEO website: http://cpheeo.gov.in. The readers are advised to refer to for further updates.

हरदीप एस पुरी
HARDEEP S PURI

आवासन और शहरी कार्य मंत्री
पेट्रोलियम एवं प्राकृतिक गैस मंत्री
भारत सरकार
Minister of Housing and Urban Affairs; and Petroleum and Natural Gas
Government of India

भारत 2023 INDIA

सत्यमेव जयते

आज़ादी का अमृत महोत्सव

MESSAGE

In 2010, the UN General Assembly recognised "the right to safe and clean drinking water and sanitation as a human right that is essential for the full enjoyment of life and all human rights." Providing safe and reliable water to our rapidly increasing urban population, in alignment with Goal 6 of the Sustainable Development Goals, will enhance the quality of life and ease of living, leading to increased productivity and economic development in the country.

India's urban water sector is under immense pressure due to the increasing population, rapid urbanisation, and climate change. To ensure sustainable and resilient urban water management, transformative changes are required. The Atal Mission for Rejuvenation and Urban Transformation (AMRUT), launched in June 2015 by the Hon'ble Prime Minister Shri Narendra Modi ji, caters to that purpose by providing water supply facilities in 500 Class-I cities. Its tremendous success and citizen acceptance led to the launch of the AMRUT 2.0 Mission which aims to make all Indian cities 'water secure' and provide functional tap connections to all urban households. The AMRUT 2.0 mission advocates for the "Drink from Tap" facility to ensure safe and reliable water for urban citizens.

This revised manual on Water Supply and Treatment will serve as a useful guide for state governments, urban local bodies, parastatal agencies, and other stakeholders for effective and efficient planning, implementation and management of water supply systems with the "Drink from Tap" facility.

I compliment the AMRUT Division, Central Public Health & Environmental Engineering Organisation (CPHEEO), Expert Committee for the preparation of this manual, as well as the support extended by Deutsche Gesellschaft für Internationale Zusammenarbeit (GIZ) GmbH and the WAPCOS study team in preparing this document.

(Hardeep S Puri)

New Delhi
03 November 2023

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कौशल किशोर
KAUSHAL KISHORE

सत्यमेव जयते

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भारत सरकार
Minister of State for Housing & Urban Affairs
Government of India

संदेश

जल आपूर्ति और जल शोधन प्रणालियों पर अदयतित नियम-पुस्तिका का लोकार्पण करते हुए मुझे बेहद हर्ष और इसके उददेश्यपरक होने की गहन अनुभूति हो रही है। यह आवासन और शहरी कार्य मंत्रालय की एक अत्यंत महत्वपूर्ण पहल है। इस यात्रा पर आगे बढ़ते हुए हमें याद रखना चाहिए कि अदयतित नियम-पुस्तिका केवल दिशानिर्देशों का संकलन मात्र नहीं है अपितु यह भारत के भविष्य की एक पथ-निर्देशिका भी है। यह हमारे नागरिकों से स्वच्छ, सुलभ और सतत जल का एक वचन है। यह एक विकसित राष्ट्र के निर्माण के प्रति हमारी प्रतिबदधता है जहां जल निर्बाध रूप से बहता है, और हर एक नल आशा की नई किरण लाता है। यह जनमानस के साहस और सामर्थ्य के प्रति कृतज्ञता है जो विपरीत परिस्थितियों में भी एक उज्जवल भविष्य के लिए प्रयासरत रहते हैं।

इस नियम-पुस्तिका के तीन प्रमुख भाग हैं। इसके भाग 'क' में इंजीनियरिंग, भाग 'ख' में संचालन और रखरखाव और भाग 'ग' में प्रबंधन को शामिल किया गया है।

इस नियम-पुस्तिका में जल आपूर्ति प्रणालियों के निर्माण और प्रबंधन की उन जटिलताओं पर गहराई से विचार किया गया है जो विविध प्रकार के क्षेत्रों में अपनाई और विकसित की जा सकती हैं। इसमें विभिन्न जल स्रोतों से जल संग्रहण के लिए नवीन कार्यनीतियाँ प्रस्तुत की गई हैं जो यह सुनिश्चित करती हैं कि संग्रहित किया गया यह जल उपभोग के उच्चतम गुणवत्ता मानकों को पूरा करता है। इसकी एक उल्लेखनीय विशेषता 'नल से पीयें जल' की सुविधायुक्त दबावयुक्त 24x7 जल आपूर्ति प्रणालियों पर ध्यान केंद्रित करना है। यह परिवर्तनकारी

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दृष्टिकोण, जिसे अक्सर कम करके आंका जाता है, लाखों लोगों के जीवन में क्रांति लाने की क्षमता रखता है। नल से सीधे स्वच्छ और सुरक्षित पेयजल की उपलब्धता एक सुविधा से कहीं अधिक बढ़कर है; यह जन स्वास्थ्य, महिला सशक्तिकरण और सामाजिक प्रगति की आधारशिला है। इसलिए इन प्रणालियों को लगन और विश्वास के साथ लागू करना हमारा दायित्व है।

'नल से पीयें जल' सुविधाओं के महत्व को रेखांकित करने के लिए, हमें उनकी प्रभावशीलता पर विचार करना चाहिए। घरों के भीतर स्वच्छ पानी की उपलब्धता का अर्थ है कि बच्चों की स्कूलों में अधिक उपस्थिति दर्ज होगी, महिलाएं आर्थिक गतिविधियों में सहभागी बन सकती हैं, और लोग दूषित जल से उत्पन्न होने वाली बीमारियों से चिंतामुक्त होकर खुशहाल जीवन जी सकते हैं। यह एक स्वस्थ और समदर्शी समाज के निर्माण की दिशा में एक ऐतिहासिक कदम है।

अमृत (अटल नवीकरण और शहरी परिवर्तन मिशन) और इसका अनुवर्ती मिशन अमृत 2.0 इस दृष्टिकोण को आगे बढ़ाने में महत्वपूर्ण भूमिका निभा रहा है। इस प्रकार की पहल, प्रत्येक नागरिक को बुनियादी सेवाएं प्रदान करने और शहरी परिदृश्य को बदलने की भारत सरकार की प्रतिबदधता को रेखांकित करती है।

आइए ! हम सभी सार्वजनिक और निजी क्षेत्रों, विशेषज्ञों और नवप्रवर्तकों के साथ मिलकर इस नियम्-पुस्तिका को एक साकार परिवर्तन का रूप प्रदान करें। आइए ! हम ज्ञान, प्रौदयोगिकी और सामूहिक इच्छाशक्ति का भरपूर उपयोग करते हुए यह सुनिश्चित करें कि प्रत्येक भारतवासी 'नल से पीयें जल' से किसी भी स्थान या परिस्थिति में सहजता से पानी पीने का आनंद ले सके।

हम सभी साथ मिलकर एक ऐसे भविष्य का निर्माण कर सकते हैं जहां जल सिर्फ एक संसाधन नहीं बल्कि जीवन, समृदधि और सम्मान का प्रतीक हो।

(कौशल किशोर)

नई दिल्ली
27 अक्तूबर, 2023

मनोज जोशी
सचिव
Manoj Joshi
Secretary

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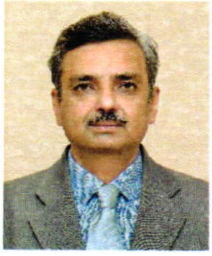

MESSAGE

India is a part of the global trend towards increasing urbanisation in which more than half of world's population is living in cities/towns. This phenomenon has been driven by factors such as industrialization, rural-to-urban migration, and economic opportunities in urban areas. Cities hold tremendous potential as engines of economic and social development. For Indian cities to become growth oriented and productive, it is essential to develop an excellent urban infrastructure by utilizing cutting-edge technology and sustainable infrastructure investments.

Water is an essential human requirement and lack of clean water has a significant influence on the health of urban people as well as the economic growth of urban areas. Therefore, it is utmost important to develop water supply infrastructure to ensure effective service delivery and sustainability.

To meet the aforesaid objective, Central Public Health and Environmental Engineering Organisation (CPHEEO), which is the technical wing of the Ministry has updated and revised the existing manual on Water Supply and Treatment as Manual of Water Supply and Treatment Systems (Drink from Tap) in three Parts – Part A-Engineering, Part B-Operation & Maintenance and Part C-Management to provide guidelines to Policy Makers, Public Health Engineers, Field Practitioners and other Stakeholders for planning, design, operation & maintenance and management of water supply systems with "Drink from Tap" facility to be taken up under various Central Missions like AMRUT 2.0 and State programs.

I would like to commend the untiring efforts of Dr. M. Dhinadhayalan, Adviser (PHEE), CPHEEO and Chariman of Expert Committee, Members of Expert Committee, AMRUT Division, Central Public Health & Environmental Engineering Organisation (CPHEEO) and the support extended by Deutsche Gesellschaftfür Internationale Zusammenarbeit (GIZ) GmbH, Germany, Government of Germany and WAPCOS study team, who were associated with the task of accomplishment of the manual for the benefit of water supply sector.

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FOREWORD

It is with immense pride and enthusiasm that I introduce the "Manual on Water Supply and Treatment Systems (Drink from Tap)" revised and updated by the Ministry of Housing and Urban Affairs. This comprehensive Manual stands as a testament to our unwavering commitment towards achieving Drink from Tap facility that will ensure efficient, sustainable, and accessible water supply for our growing urban communities.

Water, the essence of life, is a fundamental right of every individual. As our cities expand and population increases, the demand for this precious resource becomes more pressing than ever. In this context, a robust framework that encompasses every aspect of water supply and treatment is indispensable. This manual, divided into three crucial parts - Engineering, Operation & Maintenance, and Management - addresses these aspects comprehensively.

Part A: Engineering focuses on the foundation of any water supply system encompassing planning, design and implementation. By delving into detailed planning and design methodologies, technological innovations, and contemporary practices, this section equips professionals and field practitioners with the knowledge required to create efficient and resilient water supply infrastructure with decentralized approach using District Metered Areas (DMA) concept. The manual not only emphasizes conventional treatment technologies but also introduces cutting-edge technologies that have the potential to revolutionize water supply systems, ensuring sustainable service delivery and adaptability to changing urban landscapes.

Part B: Operation & Maintenance recognizes that the creation of a water supply system is only half the journey; efficient operation and vigilant maintenance are imperative to ensure its longevity. This section outlines best practices, procedures, and guidelines for maintaining the functionality of water supply systems. From routine upkeep to troubleshooting, the insights shared here will contribute to uninterrupted water supply services for urban residents by continuous monitoring and control of Non-Revenue Water (NRW) as well as monitoring and surveillance of drinking water quality using smart technologies.

Part C: Management acknowledges the multifaceted nature of water supply systems, necessitating a holistic managerial approach. By elucidating management practices, policy frameworks, and governance strategies, this section offers guidance to

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administrators and policy-makers. This part of the manual emphasised the need for Capacity Building, Asset Management and Public Private Partnership which are crucial for successful management of a Drink from Tap Water Supply System. Therefore, effective management ensures equitable distribution, financial sustainability, and the ability to adapt to dynamic urban requirements considering climate resilience.

In conclusion, the "Manual on Water Supply and Treatment Systems (Drink from Tap)" will serve as a beacon, illuminating a path towards an improved urban water management landscape.

I extend my gratitude to Dr. M. Dhinadhayalan, Adviser (PHEE), CPHEEO and Chariman of Expert Committee, Members of Expert Committee, Special invitees, CPHEEO Officials, GIZ and WAPCOS Study Team, who have contributed to this manual with the zeal to promote the practice of "Drink from Tap". It is my sincere hope that this resource becomes an indispensable companion for professionals and stakeholders engaged in the vital task of providing clean and accessible water to our urban communities.

Together, let us forge ahead in our mission to build sustainable, liveable and water secure cities, where the availability of safe water is never compromised.

(D Thara)

New Delhi

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PREFACE

Water security remains a pressing concern encompassing issues related to both quantity and quality. Contamination of surface water sources and depletion of groundwater reserves have become a significant challenge threatening long-term sustainability. Additionally, preventing contamination of drinking water from the distribution system to household underground storage sumps is a vital challenge to tackle for safeguarding public health. These challenges are crucial to address for ensuring the availability and quality of this essential resource.

The earlier Water Manual (1999) recommended that the water supply projects in urban areas shall be planned, designed and implemented to achieve 24x7 pressurised water supply system (PWSS). It also suggested to adopt residual pressure of 7m for the towns having single storey buildings, 12m for 2 storeyed buildings and 17m for 3 storeyed buildings and so on. But the Manual was grossly missing the concept of Operational Zones (OZs) and District Metered Areas (DMAs). Therefore, in the past, the Urban Local Bodies (ULBs) planned, designed and implemented water supply projects considering large size networks (large zones) without properly following the residual pressures as recommended in the earlier Manual. This led the system to shift to intermittent mode just after the commissioning of the project. At present, in almost all the towns, water supply is intermittent with a duration ranging from 2-6 hrs/day which results into contamination of water in the pipeline during non-supply hours, high Non-Revenue Water (NRW) and inequitable water supply. Due to intermittent water supply the cities are grappled with many Operation & Maintenance (O&M) and Management challenges.

Therefore, it is crucial to plan, design and implement projects by changing the conventional planning to a decentralized approach, establishing OZs and DMAs with a specific number of house service connections (HSCs), increased residual pressure and ensuring 100% metering to make the system self-sustainable. The renewed system will address the O&M and Management challenges which the systems are currently facing.

During O&M high level of NRW is an operational burden and thus monitoring and control of NRW is very crucial. Urban water service providers/utilities are unable to cover their

O&M costs due to high NRW which leads to revenue loss and increased operational costs. The constant need for repair and maintenance of aging infrastructure is essential to ensure its efficient and effective operation and maintenance of the system. Another foremost issue is lack of water quality monitoring and surveillance during O&M which is the key for sustaining the success of the project with Drink from Tap (DFT) and effective service delivery.

Urban water service providers are confronted by significant management issues due to lack of capacity and financial resources. Therefore, it is important to engage Public Private Partnership (PPP) for efficient implementation, O&M and Management of the 24x7 PWSS.

India's dream of becoming a developed nation hinges on overcoming these water-related challenges. Imagine a scenario where every household enjoys the privilege of continuous pressurised water supply with the assurance of safe drinking water directly from the tap which is the vision that drives Govt. of India initiatives like Atal Mission for Rejuvenation and Urban Transformation 2.0 (AMRUT 2.0). Achieving this vision is not just an aspiration but an imperative for a progressive, healthy and prosperous India.

Keeping in view the above the Ministry has revised the existing Manual with the focus on operationalizing the existing intermittent water supply systems to 24x7 PWSS with an objective to provide drink from tap and its ease of O&M and management. The Expert Committee constituted under the chairmanship of the undersigned with the Technical Support of GIZ in June 2020, has brought out 3 parts of the Manual to address the challenges in the planning, design, implementation, operation & maintenance and management of 24x7 PWSS.

Part A Manual (Engineering- Planning, Design and Implementation) addresses the consistent and secure supply of clean water and provides guidelines for planning, design and implementation of 24x7 water supply with Drink from Tap in urban areas based on operational zones & DMAs. It also provides guidelines for planning, design and implementation of Regional Water Supply System (RWSS) for both urban and rural areas. The prevention of contamination of water within distribution systems and household storage is emphasized along with the crucial transition from the existing intermittent water supply to 24x7 PWSS and achieving 100% metering for ensuring sustainability of 24x7 PWSS.

The Part B Manual (Operation and Maintenance) addresses challenges related to the operation and maintenance of 24x7 PWSS. It underscores the importance of maintaining aging infrastructure efficiently, offering guidance on strategies for constant repair and upkeep to extend operational life. Controlling Non- Revenue Water (NRW) through water audits and effective management is vital to reduce losses and enhance efficiency with guidance on water quality monitoring and surveillance is also included in Part B Manual.

Part C Manual (Management) emphasises the need for comprehensive reforms including legal framework, institutional strengthening, enhanced coordination, stakeholder engagement, PPP and investments in modern technology and infrastructure for emerging drink from tap projects. The need for a skilled and knowledgeable workforce to operate and maintain complex water supply systems is addressed. Financial sustainability is a key concern and provides strategies for managing finances to support effective management of water supply systems. An integrated approach is deemed crucial to ensure sustainable water services capable of meeting the growing demands of India's urban population and providing high-quality water supply particularly in the context of climate change.

We envision this revised Manual as a blueprint for the future of urban water supply and treatment systems in India. It represents our unwavering commitment to creating systems that are not only efficient but also resilient, sustainable and equitable. Our goal is clear to ensure that every urban dweller can turn on the tap and access safe, clean water without hesitation throughout day and night.

This comprehensive Manual is the outcome of tireless efforts, interdisciplinary expertise and a collective dedication to enhancing urban water supply and treatment systems across our great nation. It has been meticulously curated to encompass the ever-evolving landscape of water supply management, from cutting-edge technologies to best practices in governance and partnership models, placing us firmly on the path toward a future where every urban citizen enjoys equitable access to clean, safe and reliable drinking water.

The first Expert Committee meeting was held in March 2021. In the past two and a half years, eight (8) meetings of the Expert Committee and fourteen (14) meeting of Working Groups were held to finalize the draft of the Manual. The Expert Committee consulted with various stakeholders during National and Regional workshops on 24x7 PWSS during the preparatory phase of the Manual and also during the National Consultative Workshop on the draft Manual held on 12^{th} & 13^{th} June 2023 to get the feedback/ comments/ suggestions on the content. The Editorial Committee, constituted under the chairmanship of the undersigned, had twenty one (21) meetings between June and Oct, 2023 and deliberated and incorporated the feedbacks/ suggestions in the Manual.

I express my profound gratitude to the Ministry of Housing & Urban Affairs, Government of India for extending all support and encouragement in the revision of the Manual. I would like to express my deep gratitude to Shri Manoj Joshi, Secretary (HUA), Ministry of Housing and Urban Affairs, Government of India for his constant encouragement and lending never ending support to the team in the journey of revision of the Manual.

I would like to extend my heartfelt gratitude to Ms. D Thara, Additional Secretary & National Mission Director (AMRUT) for her inspiration, constant guidance and support without which it might not have been possible to complete this massive task of revising the Manual.

I am also privileged to express my sincere thanks to Ms. Roopa Mishra, Joint Secretary & National Mission Director (SBM), Ministry of Housing and Urban Affairs for her support in finalization of the Manual.

I would like to express my profound gratitude to GIZ for providing technical and financial support for the preparation of the Manual. My heartfelt gratitude to Shri Ernst Deoring, Former Cluster Coordinator, Shri Christian Kapfensteiner, Cluster Coordinator, Smt. Laura Sustersic, Project Director, Dr. Teresa Kerber, Project Director, Smt. Monika Bahl, Senior Advisor & Shri Rahul Sharma, Technical Advisor, GIZ for extending their support in the preparation of the Manual. They left no stone unturned to enrich the contents of the Manual by adopting participatory approach and inviting experts and all those who are working on the ground in the country as well as abroad. They flawlessly conducted all the meetings and looked after the comfort of all the members of the Committee and all those who participated in deliberations.

I also extend my gratitude to AFD for providing technical support in drafting a few chapters and to IPE Global for their contribution to enrich the Manual.

Three Working Groups were carved out of the Expert Committee to speed up the gigantic task of revision of the Manual. I would like to extend my special thanks to Dr. Sanjay Dahasahasra, Former Member Secretary, Maharashtra Jeevan Pradhikaran & Co-chairman of Working Group (Part A Manual), Dr. PN Ravindra, Former Chief Engineer, Bangalore Water Supply and Sewerage Board & Co-chairman of Working Group (Part B Manual) and Prof. V Srinivas Chary, Professor & Director of the Centre for Urban Governance, Environment, Energy and Infrastructure Development, Administrative Staff College of India (ASCI), Hyderabad & Co-chairman of Working Group (Part C Manual) for their continuous guidance, time, dedicated efforts and painstaking efforts in finalizing all three parts of the Manual and being instrumental at all stages in the journey of revision of the Manual.

I extend my heartfelt gratitude to the esteemed Members of the Expert Committee, the dedicated Editorial Committee and the invaluable Special Invitees for their selfless dedication and remarkable contributions to the Manual. Their collective expertise and diverse perspectives have significantly enriched the depth, accuracy and overall quality of the Manual. The Expert Committee's wealth of knowledge, the Editorial Committee's meticulous refinement and the specialized insights of the Special Invitees have played a pivotal role in shaping this resource into an invaluable and comprehensive guide.

I would like to extend my appreciation for Dr. Ramakant, Deputy Adviser (PHE) & Member Secretary of the Expert Committee, for his continuous support and untiring commitment towards completing the Manual. I would also like to extend my appreciation for Shri Vipin Kumar Patel and Smt. Chaitra Devoor, Assistant Advisers (PHE), CPHEEO & Member Coordinators of the Expert Committee for their restless and dedicated support in completing the assignment. I would also like to acknowledge my other colleagues from CPHEEO for extending their support.

I would like to extend my gratitude to GIZ- WAPCOS Study Team, headed by Team Leader Shri Shreerang Deshpande, Former Technical Head - Water Supply, Nashik Municipal Corporation and WAPCOS team, Shri M.A. Khan, GM (Systems), Shri Deepak Lakhanpal, Chief Engineer, Shri Rajat Jain, Chief Engineer, Engineers Shri Lalit Gupta, Shri Ishant Singhal, Shri Rishabh Chandra and Resource persons viz., Shri Himanshu Prasad, Shri Mohan Narayan Gowaikar, Shri Sandeep Bhaskaran, Dr. S.K. Sharma, Shri V.K. Gupta, Ms. Shikha Shukla Chhabra, Shri K.A. Roy, Shri Vaibhav Gupta, Shri Manmohan Prajapat, Shri Satish Kumar Kolluru and Dr. Adhirashree Vannarath, who supported GIZ study team and Shri Gaurav Bhatt for drafting the chapters. I also thank the Expert Committee members for their valuable contribution as Authors and Mentors in drafting the Manual.

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Together, let us chart a course towards a future where every urban dweller can turn on the tap and access safe, clean water without hesitation. Let us strive relentlessly to create water supply systems that are not just efficient but also resilient, sustainable and equitable. 24x7 PWSS with Drink from Tap is not just for sophistication but is a basic necessity.

(Dr. M. Dhinadhayalan)
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Chairman of the Expert Committee

New Delhi
6th November 2023

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ABBREVIATIONS

ABBREVIATIONS AND SYMBOLS

3LPE	Three Layer Polyethylene
3Ts	Tariffs, Taxes and Transfer
ABS	Acrylonitrile Butadiene Styrene
AC	Asbestos Cement
AC	Alternating Current
ACV	Air Cushion Valve
ADB	Asian Development Bank
ADC	Analog to Digital Convertor
AFD	Agence Française de Développement
AI	Artificial Intelligence
AIC	Average Incremental Cost
AMI	Advanced Metering Infrastructure
AMR	Automatic Meter Reading
AMRIT	Arsenic and Metal Removal through Indian Technology
AMRUT	Atal Mission for Rejuvenation and Urban Transformation
APFC	Automatic Power Factor Control
APHA	American Public Health Association
APT	Aquifer Pump Test
ASR	Aquifer Storage & Recharging System
ASR	Aquifer Storage & Recovery wells
ASTM	American Society for Testing and Materials
ATC	Automatic Temperature Compensated
BC	Black Cotton
BCM	Billion Cubic Meter
BDL	Below Detectable Limits
BEE	Bureau of Energy Efficiency
BEP	Best Efficiency Point
BIS	Bureau of Indian Standards
bkW	Brake Kilowatts
BMC	Bombay Municipal Corporation
BPT	Break Pressure Tank
BWRO	Brackish Water
BWSC	Bar Wrapped Steel Cylinder
BWSSB	Bangalore Water Supply and Sewerage Board

CI	Cast Iron
C2C via C	Catchment-to-Catchment-via-Consumer
CACA	Closed Air Circuit Air Cooled
CACW	Closed Air Circuit Water Cooled
CAD	Computer Aided Drawing
CBRI	Central Building Research Institute
CCT	Chlorine Contact Tank
CD	Casing for Deep Well
CDI	Capacitive Deionization
CDP	City Development Plan
CFD	Computational Fluid Dynamics
CFRO	Counterflow Reverse Osmosis
CGWA	Central Groundwater Authority
CGWB	Central Ground Water Board
CID	Cast Iron Detachable Joints
CIP	Clean In Place
CM	Casing for Medium Well
CMS	Centralised Monitoring and Control Centre
CMWSSB	Chennai Metropolitan Water Supply and Sewerage Board
CPC	Cetylpyridinium Chloride/ Hexadecyl Pyridinium Chloride
CPCB	Central Pollution Control Board
CPHEEO	Central Public Health and Environmental Engineering Organization
CPVC	Chlorinated Polyvinyl Chloride
CS	Casing for Shallow Well
CSIR	Council of Scientific & Industrial Research
CSR	Company Social Responsibility
CWBP	City Water Balance Plan
CWC	Central Water Commission
CWPRS	Central Water & Power Research Station CWR Clear Water Sump
CWT	Clear Water Tank
D	Internal Diameter
DI	Ductile Iron
DA	Dynamic Analysis
DAF	Dissolved Air Floatation

DAPRV	Direct Acting Pressure Relief Valve DBPDisinfection By-Product
DC	Direct Current
DD	Domestic Demand
DDA	Demand Dependent Analysis
DE	Diatomaceous Earth
DEM	Digital Elevation Model
DGPS	Differential Global Positioning System
DMA	District Metered Areas
DO	Dissolved Oxygen
DOL	Direct Online
DOM	Dynamic Operating Model
DP	Differential Pressure
DPCV	Dual Plate Check Valve
DPD	Di-Ethylphenylene-Di-Amine
DPR	Detailed Project Report
DSS	Decision Support System
DTP	Draft Tender Paper
DUSL	Design Useful Service Life
ES	Effective Size
EC	Electro-Chlorination
EC	Emerging Contaminant
ED	Electro-Dialysis
EDC	Endocrine Disrupting Compound
EEPROM	Electrically Erasable Programmable Read-only Memory
EF	Environmental Flows
EFW	Electric Fusion Welded
EOT	Electrically Operated Traveling Crane
EPS	Extended Period Simulation
ERW	Electric Resistance Welded
ESR	Elevated Service Reservoir
FRP	Fibre Reinforced Plastic
FBE	Fusion Bonded Epoxy
FCRI	Fluid Control Research Institute
FCV	Flow Control Valve
FD	Froude Number

FFAW	Free Flowing Artesian Well
FFAWD	Free Flowing Artesian Well Device
FGL	Finished Ground Level
FHTC	Functional Household Tap Connections
FL	Full Load
FLC	Full Load Current
FS	Flat Sheet Membranes
FSI	Floor Space Index
FSL	Full Supply Level
FTK	Field Test Kit
GI	Galvanized Iron
GRP	Glass Reinforced Plastic
GA	Genetic Algorithms
GAC	Granular Activated Chlorine
GCP	Geographic Control Point
GDWQ	Guidelines for Drinking Water Quality
GEM	Groundwater Exploration and Mapping
GEMS	Global Environmental Monitoring System
GI	Galvanized Iron
GIS	Geographic Information System
GOI	Government of India
GoM	Government of Maharashtra
GPR	Ground Penetrating Radar
GPS	Global Positioning System
GRP	Glass Reinforced Plastic
GSR	Ground Service Reservoir
GWP	Global Water Partnership
GWPI	Groundwater Potential Index
GWPZ	Groundwater Resources Potential Zone Maps
GWQM	Ground Water Quality Monitoring
GWRA	Ground Water Resource Assessment
HAA	Haloacetic Acid
HAM	Hybrid Annuity Model
HAP	Analytic Hierarchy Process
HDD	Horizontal Direction Drilling

HDET	Hand Held Data Entry Terminal
HDPE	High Density Polyethylene
HF	Hollow Fiber membranes
HFIW	High-Frequency Induction Welded
HFL	High Flood Level
HFS	Hot Finished Seamless
HGL	Hydraulic Grade Line
HGM	Hydro-Geomorphological Map
HMDA	Hyderabad Metro Development Authority
HMI	Human Machine Interface
HOT	Hand operated Traveling Crane
HSC	House Service Connection
HT	High-Tension
HTH	High Test Hypochlorite
HUG	Hydrometric Uncertainty Guidance
HW	Hazen-Williams
HWL	High Water Level
Hz	Hertz
ID	Industrial Demand
IDEMI	Institute for Design of Electrical Measuring Instruments
IDW	Inverse Distance Weighted
IEC	Information, Education & Communication
ILI	Infrastructure Leakage Index
IoT	Internet of Things
IPS	Inclined Plate Settler
IRP	Iron Removal Plant
IS	Indian Standards
ISO	International Standard Organization
ISRO	Indian Space Research Organization
IT	Information Technology
ITES	IT Enabled Services
IUWM	Integrated Urban Water Management
IUWRM	Integrated Urban Water Resources Management
IV	Isolation Valve
IWRM	Integrated Water Resource Management

IX	Ion Exchange
JICA	Japan International Cooperation Agency
KFW	Kreditanstalt für Wiederaufbau
KMC	Kolkata Municipal Corporation
KMZ	Keyhole Markup Language Zipped
KPI	Key Performance Indicators
KT	Kolhapur Type
LBF	Lake Bank Filtration
LGW	Local Ground Water
LIDAR	Light Detection and Ranging
LNF	Legitimate Night Flow
LP	Linear Programming
LPCD	Litres per Capita per Day
LPG	Linear Programming Gradient
LSL	Lowest Supply Level
LWL	Low Water Level
M&R	Maintenance and Repair
MS	Mild Steel
MAOP	Maximum Allowable Operating Pressure
MAR	Managed Aquifer Recharge
MBR	Master Balancing Reservoir
MCC	Motor Control Centre
MCL	Maximum Concentration Level
MDDL	Maximum Drawdown Level
MDG	Millennium Development Goal
MDM	Meter Data Management
MDPE	Medium Density Polyethylene
MED	Multi-Effect Distillation
MEUF	Micellar-Enhanced Ultrafiltration
MF	Microfiltration
MGD	Million Gallons per Day
MHa	Million Hectares
MIDC	Maharashtra Industrial Development Corporation
MIHAN	Multi-modal International Hub and Airport in Nagpur
MINAR	Monitoring of Indian National Aquatic Resource

MIS	Management Information System
MIU	Meter Interface Units
MJP	Maharashtra Jeevan Pradhikaran
MLD	Million Litres per Day
MLDB	Main Lighting Distribution Board
MMDB	Mono Media Deep Bed Gravity
MNF	Minimum Night Flow
MOCZ	Manganese Oxide-Coated Zeolite
MoHUA	Ministry of Housing and Urban Affairs
MoWR	Ministry of Water Resources
MSEDCL	Maharashtra State Electricity Distribution Company Limited
MSF	Multi-Stage Flash Distillation
MSL	Mean Sea Level
mWC	Meters of Water Column
NABL	National Accreditation Board for Testing and Calibration Laboratories
NAQUIM	National Aquifer Mapping and Management
NASA	National Aeronautics and Space Administration
NBC	National Building Code
NDD	Non- Domestic Demand
NDT	Non-Destructive Test
NF	Nanofiltration
NFA	Node Flow Analysis
NGT	National Green Tribunal
NHA	Node Head Analysis
NHFR	Node-Head-Flow Relationship
NIRA	National Interlinking of Rivers Authority
NIT	Nagpur Improvement Trust
NLP	Non-Linear Programming
NMC	Nagpur Municipal Corporation
NMs	Nano-Materials
NNF	Net Night Flow
NOM	Natural Organic Matter
NPSH	Net Positive Suction Head
NRLP	National River Linking Project
NRSA	National Remote Sensing Agency

NRV	Non-Return Valve
NRW	Non-revenue Water
NWDA	National Water Development Agency
NWP	National Water Policy
O&M	Operation and Maintenance
OD	Outside Diameters
ODA	Official Development Assistance
ODP	Open Drip Proof
OPVC	Oriented Polyvinyl Chloride
OT	Orthotoulidine Test
OTA	Orthotolidine Arsenite Test
OZ	Operational Zone
P	Power
P&IDs	Process/Piping and Instrumentation Diagrams
PE	Polyethylene
PN	Proctor Normal
PAC	Powdered Activated Carbon
PAP	Project Affected Person
PCR	Polymerase Chain Reaction
PDA	Pressure-Dependent Analysis
PDS	Plain Deep Well Screen
PE-AL-PE	Polyethylene-Aluminium-Polyethylene
PFAS	Poly-Fluorinated Alkyl Substances
PFC	Power Factor Controller
PFRV	Pressure and Flow Rate Reducing Valve
PLC	Programmable Logic Controller
PMC	Project Management Consultant
PMCC	Power cum Motor Control Centre
PMS	Plain Medium Well Screen
PN	Nominal Pressure
PP	Polypropylene
PPCP	Pharmaceuticals and Personal Care Product
PPP	Public Private Partnership
PPP	Pharmaceutical and Personal Care Product
PP-R	Polypropylene Random Copolymer

PRBs	Permeable Reactive Barriers
PRV	Pressure Reducing Valve
PSC	Prestressed Concrete, Cylinder or non-cylinder
PTZ	Pan Tilt Zoom
PU	Polyurethane
PVC	Poly-Vinyl Chloride
PVDF	Poly-Vinylidene Fluoride
PVRV	Pressure Vacuum Relief Valve
PW	Present Worth
RC	Reinforced Concrete
RBF	River Bank Filtration
RCC	Reinforced Cement Concrete
RCW	Recycled Water
RC-Wells	Radial Collector Wells
RDS	Ribbed Deep Well Screen
RF	Radio Frequency
RFP	Request for Proposal
RM	Consumer Relations Management
RMS	Ribbed Medium Well Screen
RO	Reverse Osmosis
ROI	Return on Investment
ROVs	Remotely Operated Vehicles
RPMs	Revolutions per Minute
RRWSS	Rural Regional Water Supply Scheme
RTUs	Remote Terminal Units
RWH	Rain Water Harvesting
RWSS	Rural Water Supply Scheme
SCADA	Supervisory Control and Data Acquisition
SDB	Sludge Drying Bed
SDG	Sustainable Development Goal
SDI	Silting Density Index
SDR	Standard Dimension Ratio
SEC	Specific Energy Consumption
SEZ	Special Economic Zone
SIV	System Input Volume

SLB	Service Level Benchmark
SOM	Synthetic Organic Matter
SOP	Standard Operating Procedure
SOR	Surface Overflow Rate
SPDP	Screen Protected Drip Proof
SPV	Solar Photo Voltaic
SPV	Special Purpose Vehicle
STP	Sewage Treatment Plant
SV	Sluice Valve
SWD	Side Water Depth
SWM	Solid Waste Management
SWOT	Strengths Weaknesses Opportunities Threats
SWRO	Seawater Reverse Osmosis
TBL	Triple Bottom Line
TCLP	Toxicity Characteristics Leaching Procedure
TDS	Total Dissolved Solids
TEFC	Totally Enclosed Fan Cooled
TESWC	Totally Enclosed Self Water Cooled
TETV	Totally Enclosed Tube Ventilated
TFC	Thin Film Composite
TGW	Treated Ground Water
THMs	Trihalomethanes
TMP	Transmembrane Pressure
TOF	Time of Flight
TSS	Total Suspended Solids
TSW	Treated Surface Water
TTRO	Tertiary Treatment RO
UC	Uniformity Coefficient
UN SDG	United Nations' Sustainable Development Goal
UF	Ultrafiltration
UFW	Unaccounted for Water
ULBs	Urban Local Bodies
UN	United Nations
UNICEF	United Nations International Children's Emergency Fund
UPVC	Unplasticized Polyvinyl Chloride

UV	Ultraviolet
UWTP	Used Water Treatment Plants
VC	Vapour Compression
VCB	Vacuum Circuit Breaker
VFD	Variable Frequency Drive
VOC	Volatile Organic Compounds
VSD	Variable Speed Drive
VT	Vertical Turbine
WBE	Wastewater-Based Epidemiology
WDN	Water Distribution Networks
WDS	Water Distribution System
WHO	World Health Organization
WL	Water Level
WQI	Water Quality Index
WRC	Water Research Council
WRD	Water-Resources Division
WRIS	Water Resource Information System
WRM	Water Resources Management
WSP	Water Safety Plan
WTM	Water Transmission Mains
WTN	Water Transmission Network
WTP	Water Treatment Plant
WWAP	World Water Assessment Programme
ZBR	Zonal Balancing Reservoir
ZVV	Zero Velocity Valve

GLOSSARY

GLOSSARY

24x7 Pressurised Water Supply System, a system having continuous pressurised water supply with Drink from Tap facility.

A

Adsorption, is a physical process in which dissolved molecules or small particles in water (the adsorbate) are attracted and become attached to the surface of something larger (the adsorbent)

Aeration, is a process of treatment that consists of passing large amounts of air through water and then venting the air outside. The air causes the dissolved gases or volatile compounds to release from the water. The air and the contaminants released from the water are vented

Air Valves, are hydromechanical devices with an internal float mechanism designed to release trapped air during filling and operation of a piping system

Air Vessel, is used to compensate for pressure fluctuations and as safety device to avoid surge pressure

Algae, is the plural form of the word alga, which in Latin means "seaweed." and are defined as a group of predominantly aquatic, photosynthetic, and nucleus-bearing organisms that lack the true roots, stems, leaves, and specialized multicellular reproductive structures of plants

Algicides, are chemical compounds whose active ingredients kill algae and/or prevent it from growing in water

Alkalinity, Capacity of a Water to neutralise acids. It is usually expressed in milligrams per litre of equivalent calcium carbonates

Automatic Meter Reading, is a technology used to automatically collect consumption, diagnostic and status data through water metering devices. The AMR then transfers this datato a central database for billing, troubleshooting and analysis

Anti-Vacuum Valve, is a very special type of air valve**.** Its primary function is to prevent the formation of vacuum in large diameter water mains, which might cause line collapse under such conditions of flow as may result from too rapid a closure of an upstream head gate or shut down valve, or ordinary emptying of a pipeline

Aquifer, is a geological formation that is permeable enough to transmit sufficient quantities ofwater to support the development of water wells.

Aquifer Vulnerability Index, the aquifer is vulnerable to surface contaminants and the Aquifer Vulnerability Index is a method of assessing the vulnerability of aquifers to surface contaminants. It is assessment of risk accumulated with groundwater resources

Automation, is the use of technology to control a system or process without human intervention. In the context of water supply, automation can be used to control a variety of aspects of the water distribution system, including Pumping, Valves etc.

B

Benchmark, is the level of supply and the quality of water that a consumer is entitled to get.

Borewell, a deep narrow well for water drilled into ground & has pipe fitted as a casing in theupper part of the borehole and a pump to draw water to the surface

Branched Transmission Main, is a branch main that is off taking from the transmission mainfor coverage of enroute habitations.

Bulk-Meter, is a large meter that is usually fitted to pipes to measure bulk water quantity delivered to elevated service reservoirs and is also used in water auditing and leak detection purposes

Break Pressure Tank, to break the hydrostatic pressure, a tank is specially built which is known as a break pressure tank. It will be located at the highest elevation of the transmissionpipeline and is required to manage the water pressures that will be generated in the operationof the transmission pipeline.

Brine, water saturated or strongly impregnated with common salt

Butterfly Valve, a valve consisting of a rotating circular plate or a pair of hinged semicircular plates, attached to a transverse spindle and mounted inside a pipe in order to regulate or prevent flow. These valves are used where space is limited and can be used for throttling or regulating flow as well as in the full open and fully closed position. The pressure loss through a butterfly valve is small in comparison with the gate valve

C

Carcinogenic, having the potential to cause cancer

Check Dam, is a small, sometimes temporary, dam constructed across a swale, drainage ditch, or waterway to counteract erosion by reducing water flow velocity

Chloramines, (also known as secondary disinfection) are disinfectants used to treat drinkingwater and they are most commonly formed when ammonia is added to chlorine to treat drinking water, provide longer-lasting disinfection as the water moves through pipes toconsumers

Chlorination, Water chlorination is the process of adding chlorines or chlorine compounds such as sodium hypochlorite to water. Chlorination is used to prevent the spread of water borne diseases

Chlorine Residual, is the low-level amount of chlorine remaining in the water after a certain period or contact time after its initial application. It constitutes an important safeguard against the risk of subsequent microbial contamination after treatment—a unique and significant benefit for public health

Chlorinator, is a device to apply or to deliver a chlorine disinfectant to water at a controlled rate

Canadian Investment Regulatory Organization, regulates the mutual fund dealers that invest in water funds. These funds invest in water infrastructure companies and other water-related businesses. This can help to make water investment more accessible to individual investors.

Coagulant, is a chemical that is used to remove suspended solids from drinking water. They are made up of positively charged molecules, which help to provide effective neutralization of water

Coagulation, is the chemical water treatment process used to remove solids from water, by manipulating electrostatic charges of particles suspended in water. This process introduces small, highly charged molecules into water to destabilize the charges on particles, colloids, or oily materials in suspension

Cold Desert, is an arid habitat with an annual rainfall of less than 25 cm. They have a temperate climate with scorching summers and chilly winters because they are situated at a high latitude.

Confined Aquifer, is an aquifer below the land surface that is saturated with water. Layers of impermeable material are both above and below the aquifer, causing it to be under pressure so that when the aquifer is penetrated by a well, the water will rise above the top of the aquifer

Contamination, is defined as any substance added to water that degrades its quality. Water bodies include lakes, rivers, oceans, aquifers, reservoirs and groundwater

Consumer Survey, is a source to obtain information about consumer satisfaction levels with existing water quality and services and their opinions and expectations regarding new water quality and services

Control Valve, is a valve used to control fluid flow by varying the size of the flow passage as directed by a signal from a controller. This enables the direct control of flow rate and the consequential control of process quantities such as pressure, temperature, and liquid level

Cryptosporidium, Cryptosporidium parvum is a waterborne parasite encased in a leathery shell, (or oocyst), and causes severe flu-like symptoms when ingested.

City Water Balance Plan, is a document that describes the water resources of a city, including the sources of water, the demand for water, and the ways in which water is used and managed. The CWBP is used to identify the water supply and demand gaps in a city and to develop strategies to close these gaps.

City Development Plan, sets out how best the city can enable growth and investment over the years.

Communication Technologies, Communication technologies are used in water supply for a variety of purposes, including Monitoring and control of water infrastructure, Asset management, Customer service, Emergency response and Research and development.

D

Dual Water Distribution System, for coastal cities and new layouts of water scarce cities consist of two independent pipe networks with separate treatment, pumping and storage system to supply different grade of water to consumers.

Debottlenecking, is defined as the process of pinpointing specific areas in plant equipment or the workflow configuration that limits the flow of product. By optimising plant operations, overall capacity and/quality can be improved

Digital Terrain Modelling, is a mathematical representation (model) of the ground surface, most often in the form of a regular grid, in which a unique elevation value is assigned

Digitalization, describes the pure analog-to-digital conversion of existing data and documents.
Digital Twin, is a virtual representation of an object or system that spans its lifecycle, is updated from real-time data, and uses simulation, machine learning and reasoning to help decision making

Disaster, is an event whose timing is unexpected and whose consequences are seriously destructive

Disinfection, means the removal, deactivation or killing of pathogenic microorganisms. Microorganisms are destroyed or deactivated, resulting in termination of growth and reproduction

Distillation, is a process that relies on evaporation to purify water. Contaminated water is heated to form steam. Inorganic compounds and large non-volatile organic molecules do not evaporate with the water and are left behind. The steam then cools and condenses to form purified water

District Metered Area, is defined as a discrete part of a water distribution network. It is usually created by closing boundary valves or by permanently disconnecting pipes to neighbouring areas

Detailed Project Report, consists of detailed data, design drawings and estimate of a prospective project

Drink from Tap, continuous pressurised water supply system to ensure water quality for drinking, cooking, washing, etc. made available to consumer tap.

Drones, is a flying robot that can be remotely controlled or fly autonomously using software- controlled flight plans in its embedded systems used for various purpose in water sector and other areas

Distribution System, A water distribution system is a network of pipes, pumps, and other infrastructure that delivers water from a treatment plant to homes and businesses.

E

Electrical Conductivity, is a measure of the capability of water to pass electrical flow. This ability directly depends on the concentration of conductive ions in the water. These conductive ions originated due to inorganic materials such as chlorides, alkalis, carbonate and sulphide compounds and dissolved salts. The unit of EC is milli-Siemens per meter (mS/m)

Electro-chlorination, is the process of producing hypochlorite by passing electric current through salt water. This disinfects the water and makes it safe for human use, such as for drinking water or swimming pools

Electro-dialysis, is a process controlled by an electric field gradient that allows the separation of minerals from feed water solution. It moves dissociated ions through ion-permselective membranes and forms two different flows - desalinated flow called dilute and a concentrated flow called concentrate (brine)

Electrofusion, is a method of joining MDPE, HDPE and other plastic pipes using special fittings that have built-in electric heating elements which are used to weld the joint together

Emerging Contaminants, are those which have not previously been detected through water quality analysis, or have been found in small concentrations with uncertainty as to their effects. The risk they pose to human or environmental health is not fully understood

Energy Audit, is an inspection survey and an analysis of energy flows for energy conservation and includes a process or system to reduce the amount of energy input into the system without negatively affecting the output

Elevated Service Reservoirs, are constructed, where water is to be supplied at elevated height (less than the level of ESR) or where the distance is large and topography is undulating

Estuary, it is a partially enclosed coastal body of brackish water with one or more rivers or streams flowing into it and with a free connection to the open sea

F

Filter Console, provides continuous and discrete controls that are necessary for a typical surface or bulk filter in a water treatment plant

Filter Sand, Quartz sand, silica sand, anthracite coal, garnet, magnetite, and other materials may be used as filtration media. Silica sand and anthracite are the most commonly used types

Filtration, is the process in which solid particles in a liquid or gaseous fluid are removed by the use of a filter medium that allows the fluid to pass through while retaining the solid particles.It may mean the use of a physical barrier, chemical, and/or a biological process

Floating Reservoirs, during peak demand in the distribution system, water from the source as well as from the storage reservoir will be supplied. The storage reservoir under this condition is called Balancing Reservoir. Balancing reservoir is also called floating reservoir

Flocculation, is a water treatment process where solids form larger clusters, or flocs, to be removed from water. This process can happen spontaneously, or with the help of chemical agents. It is a common method in the purification of drinking water

Flow Control Valve, is designed to maintain a constant pre-set maximum flow regardless of fluctuating demand or varying system pressure. Flow limiting is required at the outlets from main systems to consumers like secondary systems (main line to hydrant line; hydrant line to distribution line), reservoirs, etc.

Flow-meters, are critical instruments in water treatment plants, providing accurate measurement and control of water flow to achieve efficient treatment processes, meet regulatory requirements, conserve water, and maintain optimal plant performance

Flumes, A device used to measure the flow in an open channel. The flume narrows to a throat of fixed dimensions and then expands again. The rate of flow can be calculated by measuring the difference in head (pressure) before and at the throat of the flume

Foot Valve, is a type of check valve that is typically installed at a pump or at the bottom of a pipe line (hence the name). Foot valves act like ball check valves, but have an open end with a shield or screen over it to block debris from entering the line.

G

Geographic Information System, is an effective tool for storing, managing, and displaying spatial data often encountered in water resources management. The application of GIS in water resources is constantly on the rise

Globe Valve, is an instrument used to stop and/or control the flow of fluids in a pipeline. It works by halting the flow of a fluid through a pipe. The name globe comes about due to the valve's cylindrical shape. There are usually two halves of the body within the globe valve that are separated by an internal baffle

Ground Penetrating Radar, is a geophysical locating method that uses radio waves to capture images below the surface of the ground in a minimally invasive way

Gravity Transmission Main, Gravity water systems use gravity to transport water from the source to the user through a pipe network.

Groundwater, is water that exists underground in saturated zones beneath the land surface

Groundwater Table, the top of the subsurface ground-water body, the water table, is a surface, generally below the land surface, that fluctuates seasonally and from year to year in response to changes in recharge from precipitation and surface-water bodies

Guniting, is a technique of applying mortar or concrete to a surface with a spray cannon during construction

H

Halogen, elements are fluorine (F), chlorine (Cl), bromine (Br), iodine (I), astatine (At) and tennessine (Ts). Because of their great reactivity, the free halogen elements are not found in nature. Halogen reacts to a small extent with water, forming acidic solutions with bleaching properties. They also undergo redox reactions with metal halides in solution, displacing less reactive halogens from their compounds

Hazen William Co-efficient (C), is usually considered independent of pipe diameter, velocity of flow and viscosity. However, to be dimensionally consistent and to be representative of friction conditions, it must depend on relative roughness of pipe and Reynold's number

Head Works, is a civil engineering term for any structure at the head or diversion point of a waterway. When dam is constructed across a river to form a storage reservoir, it is known as storage head work. It stores water during the period of excess supplies in the river and releases it when demand overtakes the available supplies

Hydrogeology, the study of the occurrence distribution, and movement of underground water

Hydrogeomorphic Map, Hydro-geomorphological Maps incorporate relationship of geomorphic units with their groundwater potential as interpreted from landform characteristics as well as sub-surface geology

Hydraulic Modelling, is a collection of mathematical equations that give a simple representation of reality. They estimate flow, water level and velocity in river channels and pipe networks. A hydraulic model can make these calculations and simulate infrastructure performance. Visibility into deviations from forecast, Demand forecasting and other forecast models are critical tools that can help water utilities plan for the future

I

Intermittent Water Supply, defined as piped water supply service that is available to consumers less than 24 hours per day. In an IWS situation, the consumers usually secure their water supply through the use of ground or roof tanks, where water is stored during the length of time that the supply is provided

IOT, short form of Internet of things describes the network of physical objects— “things”— that are embedded with sensors, software, and other technologies for the purpose of connecting and exchanging data with other devices and systems over the internet

Ion Exchange, systems are used for efficient removal of dissolved ions from water. Ion exchangers exchange one ion for another, hold it temporarily, and then release it to a regenerant solution. In an ion exchange system, undesirable ions in the water supply are replaced with more acceptable ions

Isoheytal Map, map depicting characteristics of equal precipitation amounts recorded during a specific time period

Isotopes, atoms with same number of protons but different number of neutrons.

Integrated Water Resources Management, is a process that promotes the coordinated development and management of water, land and related resources in order to maximize economic and social welfare in an equitable manner without compromising the sustainability of vital ecosystems

Integrated Urban Water Resources Management, is a process that promotes the coordinated development and management of urban water, urban land and related urban resources in order to maximize economic and social welfare in an equitable manner without compromising the sustainability of vital ecosystems.

K

K value in pipe, Resistance coefficient K is proportional coefficient between pressure drop (head loss) and square velocity of fluid flowing through valves and fittings like an elbow, bend, reducer, tee, pipe entrance, and pipe exit

Kinetics of Disinfection, the rate of destruction of micro-organisms has been expressed by a first order reaction referred to as “Chick's law.” Chick's law states that the rate of bacterial destruction is directly proportional to the number of organisms remaining at any time

L

Litres per Capita per Day, the level of water supply means actual quantity of the drinking water in litre per capita per day (lpcd) provided to the population

LoRa WAN, stands for Long Range Wide Area Network. It is a low-power wide-area networking (LPWAN) technology that is designed to connect battery-powered devices over long distances with same uses as NB IOT.

M

Manholes, are the commonly used maintenance utility underground structures to provide access to installed pipelines for inspection and cleanout. It is a vital component of the water supply and sanitary system, the basic underground utilities

Manometers, instruments for measuring the pressure acting on a column of fluid, consisting of a U-Shaped tube of a liquid in which a difference in the pressures acting in the two arms of tube causes the liquid to reach different heights in the two areas

Main Balancing Reservoirs, are larger than zonal balancing reservoirs and are located at the headwaters of a water distribution system. They are used to regulate water pressure and distribution for the entire system.

Membrane Desalination, is the process by which salt and minerals are removed from water solution when it passes through a semipermeable membrane

Managed Aquifer Recharge, is a water management approach that can be used to maximize natural storage and increase water supply system resilience during periods of low flows and high seasonal variability

N

NAQIUM, represents National Project on Aquifer Management implemented by Central Ground Water Board (CGWB) for the Mapping of Aquifers in India

NB-IOT, Narrowband IoT is a low-power wide-area network (LPWAN) radio technology standard developed by 3GPP for cellular network devices and services. NB-IoT is used in a variety of IoT applications, including Asset Tracking, Smart Metering, Environmental Monitoring, Industrial automation and Smart city applications.

Non-Revenue Water, is water that has been produced and is "lost" before it reaches the customer. Losses can be real losses (through leaks, sometimes also referred to as physical losses) or apparent losses (for example through theft or metering inaccuracies)

O

Over-exploited Unit, are those units where groundwater abstraction substantially exceeds (more than 100%) the annually replenishable ground water

Ozone, is produced when oxygen (O_2) molecules are dissociated by an energy source into oxygen atoms and subsequently collide with an oxygen molecule to form an unstable gas, ozone (O_3), which is used to disinfect water and wastewater

Ozonation, is a type of advanced oxidation process, involving the production of very reactive oxygen species able to attack a wide range of organic compounds and all microorganisms

P

Parastatals, refers to a government entity or agency that operates independently of the formal government structure, but is ultimately accountable to the government such as semi- autonomous, state-owned, quasi-governmental, public enterprise, government-owned corporation, and statutory corporation.

Pathogenic Organism, includes bacteria, viruses or cysts, capable of causing diseases (typhoid, cholera, dysentery) in a host (such as a person). There are many types of organisms which do not cause disease

Peak Factor, is typically expressed as a ratio, or peaking factor, dividing the peak water use by the average daily water use. These peaking factors are then used to calculate maximum month, maximum day and peak hour water use conditions

PERT Diagram, stands for program evaluation and review technique diagram. It provides a visual representation of a project's timeline and breaks down individual tasks. These charts are similar to Gantt charts, but structured differently. This diagram consists of a few steps to get you from a project start date to end date

Pesticides, are chemical compounds that are used to kill pests, including insects, rodents, fungi and unwanted plants (weeds)

pH, is a measure of how acidic/basic water is. The range goes from 0 to 14, with 7 being neutral. pHs of less than 7 indicate acidity

pH Meter, is an instrument used to measure acidity or alkalinity of a solution - also known as pH. pH is the unit of measure that describes the degree of acidity or alkalinity. It is measured on a scale of 0 to 14

Pneumatic System, is a collection of interconnected components using compressed air to do work for automated equipment.

Potable Water, is defined as water that is suitable for human consumption (i.e., water that can be used for drinking or cooking). The term implies that the water is drinkable as well as safe

Pressure Filter, is the process of separating a suspended solid such as a precipitate from the liquid in which it is already suspended by straining it – under pressure – through a porous medium that can be penetrated easily by liquid

Public-Private Partnership, model is a partnership between the public sector and the private sector for the purpose of delivering a project or a service traditionally provided by the public sector.

Pump House, a building containing pumping equipment, thus, a building containing pumping equipment to provide the water supply from a well, spring, creek, or pond, water treatment plant, clear water reservoir, service reservoirs, etc.

R

RADAR, a device that sends out radio waves for detecting and locating an object by the reflection of the radio waves and that may use this reflection to find out the position and speed of the object

Radiography, X-ray, also referred to as radiography, enables NDT technicians to analyse the interior and exterior structure of pipes without having to alter or damage any components. X- ray inspections require having access to two sides of the pipe – one side to transmit radiation, and one side to record it

Recycled Water, water reuse (also commonly known as water recycling or water reclamation) reclaims water from a variety of sources then treats and reuses it for beneficial purposes such as agriculture and irrigation, potable water supplies, groundwater replenishment, industrial processes, and environmental restoration

Resilient, resilience is the ability of social-ecological systems to weather and recover from shocks while remaining adaptable to an uncertain future, and “water resilience” refers to those characteristics in a water system

Rejuvenation, restoration to its original or near original like structure

Remote Sensing, is the process of detecting and monitoring the physical characteristics of an area by measuring its reflected and emitted radiation at a distance (typically from satellite or aircraft). Special cameras collect remotely sensed images, which help researchers "sense" things about the Earth

River Basin, is the portion of land drained by a river and its tributaries. It encompasses all of the land surface dissected and drained by many streams and creeks that flow downhill into one another

Robotic, systems are defined as systems that provide intelligent services and information by interacting with their environment, including human beings, via the use of various sensors, actuators and human interfaces.

S

Safe Yield, is defined as the maximum rate of withdrawal that can be sustained by an aquifer without causing an unacceptable decline in the hydraulic head or deterioration in water quality in the aquifer

Sedimentation, is the process of separating small particles and sediments in water. This process happens naturally when water is still because gravity will pull the heavier sediments down to form a sludge layer. However, this action can be artificially stimulated in the water treatment process

Simulation, is the imitation of the operation of a real-world process or system over time

Specific Capacity of a Well, is defined as the pumping rate divided by drawdown at some time after pumping was started

Satellite Images, are images of Earth collected by imaging satellites operated by governments and businesses around the world

Sanitation, system includes the capture, storage, transport, treatment and disposal or reuse of human excreta and wastewater

SCADA, short form of Supervisory control and data acquisition is a system of software and hardware elements that allows organisations to control processes locally or at remote locations, monitor, gather, and process real-time data

Service Hazards, service hazards in water supply are the potential risks to human health and the environment that can occur during the delivery of water to homes and businesses. These hazards can be caused by a variety of factors, including Contamination, Physical hazards, biological hazards and Chemical hazards.

Seepage, the flow of water or any fluid through the soil or ground is called seepage.

Sluice Valves, use a gate or a wedge-shaped disc to control and regulate the flow. This gate runs perpendicular to the flow of fluids into or out of the pipeline. The valve opens by lifting the gate out of the path of the fluids and enabling it to flow

Soil Resistivity, it is the measure of soil’s capability to oppose, resist, and reduce the flow of electric current through it. Soil Resistivity is determined by its content of electrolytes which consist of moisture, minerals, and dissolved salts

Solar Stills, is a device to desalinate impure water like brackish or saline water. It a simple device to get potable/fresh distilled water from impure water, using solar energy as fuel, for its various applications in domestic, industrial and academic sectors

Solar Pump, is an application of photovoltaic technology which converts solar energy into electricity to run the pumping system thereby replacing erratic grid supply and pollution- causing diesel-powered versions

Solar Panels, are those devices which are used to absorb the sun's rays and convert them into electricity or heat

Spring, is a natural opening in the ground where water emerges & flows directly from the aquifers to earth surface

Stakeholders, anyone who can affect or be affected by the urban water service delivery

Static Head, sometimes referred to as the pressure head, is a term primarily used in Hydraulics to denote the static pressure in a pipe, channel, or duct flow

Storage Sumps, is an underground (or partially underground) tank that is usually used for large water tank storage and can be built using cement-like materials.

Submersible Pump, is a pump that can be fully submerged in water. The motor is hermetically sealed and close-coupled to the body of the pump. A submersible pump pushes water to the surface by converting rotary energy into kinetic energy into pressure energy

Surge Tank, is a standpipe or storage reservoir at the downstream end of a closed aqueduct, feeder, dam, barrage pipe to absorb sudden rises of pressure, as well as to quickly provide extra water during a brief drop in pressure

Surge-Shaft, is a structure provided at the end of headrace tunnel or pipe to account for water hammering effect in the pipe at its downstream

T

Tariff, is a price assigned to water supplied by a public utility through a piped network to its customers

Total Dissolved Solids, is a measure of the dissolved combined content of all inorganic and organic substances present in a liquid in molecular, ionized, or micro-granular (colloidal sol) suspended form. TDS are often measured in parts per million (ppm). TDS in water can be measured using a digital meter

Telemetry, is the automatic measurement and wireless transmission of data from remote sources. In general, telemetry works in the following way, Sensors at the source measure either electrical data, such as voltage and current, or physical data, such as temperature and pressure

Total Organic Carbon, within water treatment is referring to the total amount of organic carbon found in water

Transformer, device that transfers electric energy from one alternating-current circuit to one or more other circuits, either increasing (stepping up) or reducing (stepping down) the voltage

Turbidity Meter, technically known as nephelometers – emit light and measure the amount scattered by particles in the sample. The units depend on the wavelength of the light and the angle of the detector(s); the most common units are Nephelometric Turbidity Units (NTU) or Formazin Nephelometric Units (FNU)

Turbine Pump, is a class of centrifugal pump which uses turbine-like impellers with radially oriented teeth to move liquids. Turbine pumps are commonly used in installations which require high head, low flow, and compact design. A vertical turbine pump commonly removes water from an underground well or reservoir.

U

UG-Tank, Underground tank is meant to store treated disinfected water supplied from the ULBs for use by the residences of the building

ULBs, Urban Local Bodies mean Municipal Corporation, Municipality or Town that administers or governs a city or a town of specified population Panchayat

Ultrafiltration, is a variety of membrane filtration in which hydrostatic pressure forces a liquid against a semi permeable membrane. Suspended solids and solutes of high molecular weight are retained, while water and low molecular weight solutes pass through the membrane

Ultrasonic, vibrations of frequencies greater than the upper limit of the audible range for humans that is, greater than about 20 kilohertz. The term sonic is applied to ultrasound waves of very high amplitudes

Ultrasonic Pulse Velocity (UPV), it is an in-situ, non-destructive test to check the quality of concrete & natural rocks

Ultrasonic Water-meters, it comes with two transducers which trigger sound waves. Sound waves determine the velocity of a water flowing in a pipe. Under no flow conditions, the frequencies of an ultrasonic wave transmitted into a pipe and its reflections from the fluid are the same

Unconfined Aquifers, are those that rock is directly open at the surface of the ground and groundwater is directly recharged, for example by rainfall or snow-melt. The Upper water surface of unconfined aquifer is at atmosphere pressure

Up-Flow Filter, Upflow units contain a single filter medium–usually graded sand. The finest sand is at the top of the bed with the coarsest sand below. Gravel is retained by grids in a fixed position at the bottom of the unit. The function of the gravel is to ensure proper water distribution during the service cycle.

V

Vertical Turbine Pumps, are centrifugal pumps, also known as the vertical pump, deep well, or line shaft pump. They are designed to move water from underground wells or reservoirs

VFD Pump, short form of variable frequency drive is a type of drive that controls the speed, of a non-servo, AC motor by varying the frequency of the electricity going to that motor. VFDs are typically used for applications where speed and power are important.

W

Wastewater, is any water that has been adversely affected in quality by anthropogenic influence and comprises liquid waste discharged by domestic residences, commercial properties, industry, and/or agriculture and can encompass a wide range of potential contaminants and concentrations

Water Audit, is a systematic process of objectively obtaining a water balance by measuring flow of water from the site of water withdrawal or treatment, through the distribution system, and into areas where it is used and finally discharged

Water Treatment, refers to a process, device, or structure used to improve the physical, chemical, or biological quality of the water in a public water system

Water Distribution Networks, is a part of water supply network with components that carry potable water from a centralized treatment plant or wells to consumers to satisfy residential, commercial, industrial and firefighting requirements

Water Quality Index, provides a single number that expresses the overall water quality, at a certain location and time, based on several water quality parameters. The objective of WQI is to turn complex water quality data into information that is understandable and usable by the public

Z

Zonal Balancing Reservoirs, are typically smaller than main balancing reservoirs and are located within a specific zone of a water distribution system. They are used to regulate water pressure and distribution within that zone.

Zero Velocity Valve, consists of a spring-loaded closing disc for stopping reverse flow in case of failure of pumps. It is enclosed in an outer shell. As the forward velocity of water reduces to near zero, the springs close the disc on the seat and breaks the returning water column to prevent positive pressure surge

Zeolite, is a mineral that can form into a variety of structures made of arrays of aluminium

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1. INTRODUCTION

Safe drinking water is most essential for the human health and well-being of people in India. Contamination of drinking water gives rise to many water borne diseases like cholera, diarrhoea, dysentery, hepatitis A, typhoid and polio. Adequate or appropriately managed water and sanitation services help to avoid preventable water borne diseases and health risks.

India's urban water sector is under immense pressure due to the increasing population, rapid urbanization and water scarcity. Inefficient management, aging infrastructure, contamination, and climate change further exacerbate the situation. ULBs are focusing on creation of infrastructure rather than improving service levels to ensure a sustainable and resilient urban water sector, transformative changes are required.

The objective of this revised manual is to provide comprehensive guidelines for effective planning, design, implementation, O&M and management of 24x7 Pressurised Water Supply System (24x7 PWSS) with drink from tap meeting drinking water quality standards, IS 10500:2012. This revised Manual aims at presenting a detailed analysis of the challenges faced by the Indian urban water sector and outlines strategies to achieve a successful transformation. It intends to serve as a guide to the field engineers, practitioners, administrators, managers engaged in the water supply sector. The Manual comprises of three parts Part A (Engineering), B (O&M) and C (Management).

The Part A of the Manual provides comprehensive guidelines for planning, investigation, design and implementation of water supply schemes to achieve 24x7 PWSS with *drink from tap* by converting existing intermittent water supply systems as well as planning, design and implementation of new water supply systems in urban areas. It also provides guidelines for planning, design and implementation of Regional Water Supply Schemes (RWSS) for both Urban and Rural areas.

The Executive Summaries of Part B and Part C Manuals are provided in the respective Parts of the manual.

2. PRESENT SCENARIO OF WATER RESOURCE AVAILABILITY

Table 1: Sector wise water demand (BCM)

Sector	2010	2025	2050
Irrigation	688	910	1072
Drinking water	56	73	102
Industry	12	23	63
Energy	5	15	130
Others	52	72	80
	813	1093	1447

Source: https://www.statista.com/statistics/report-content/statistic/1111839

India is a home for 17% of the world's population but has only 4% of the world's freshwater resources. Every year India receives about 4,080 billion cubic meters (BCM) of water as annual renewable water resources. From the surface water and replenishable groundwater, 1,999 BCM water is available annually but only 60% of it can be beneficially used. Thus, India's total available water resource is 1,128 BCM out of which 690 BCM is surface water and 438 BCM is in the form of groundwater. The surface and groundwater approximately contribute 61% and 39% of total availability. In India, 90% of flow in the rivers occurs in 4 months of monsoon and 50% of this occurs in just 15 rainy days. As per the estimate the water resources availability will be 1191 BCM, whereas demand for the water will be 1447 BCM by the turn of the year 2050 for all users like irrigation, drinking water, industry, energy and others. The sector wise water demand for different years is shown in Table 1. There is a gap of about 256 BCM. Hence, it is essential to bring reforms in the water sector with focus on conservation of water though recycling of wastewater, rain water harvesting and control of NRW etc.

As per 2011 census the coverage of pipe water supply in urban areas was 71% NITI Aayog (2019) stated that 93% of India's urban population had access to basic water supply. The AMRUT Mission was launched by the Ministry of Housing and Urban Affairs (MoHUA) in 2015. Universal piped water supply coverage was the objective under the Atal Mission for Rejuvenation and Urban Transformation (AMRUT) in 500 cities of India. As of November 2023, 1.73 Crore new tap connections have been provided under AMRUT. AMRUT 2.0 was launched by MoHUA in October 2021 with an objective to provide water security and 100% functional tap connections in all cities and towns in the country with the target of 2.68 Crore connections till 2026.

One of the objectives of the AMRUT 2.0 is to provide 24x7 pressurized water supply system (24x7 PWSS) with the drink from tap facility in at least 1 zone or 2000 connections in 500 AMRUT cities. There lies a great challenge ahead to supply continuous water supply to every household with functional water tap.

Many cities in India are moving towards 24x7 PWSS with drink from tap facilities. While cities such as Puri, Malkapur, Alnawar, Kundagol, Thirthahalli, Indi etc. have successfully 100% converted their intermittent water supply system to 24x7 PWSS, some major cities are in the process of upscaling 24x7 PWSS. City of Vishakhapatnam has been implementing 24x7 PWSS for 3 Lakh population. Coimbatore and Nagpur also commissioned their supply to 24x7 PWSS partly. In Puri, Drink from Tap (DFT) is practiced and the Government of Odisha has embarked on its journey of Drink from Tap in 23 towns.

3. MAJOR TECHNICAL CHALLENGES IN URBAN WATER SUPPLY SYSTEMS

The Urban water supply systems are essential for sustaining urban growth and economic development. However, Indian cities are grappling with a range of challenges that impede the effective service delivery of clean and adequate water to their residents, which are as follows:

a) Water security including Quantity and Quality
b) Conversion of Intermittent water supply to 24x7 PWSS with Operational Zones (OZs) and District Metered Areas (DMAs)
c) Strategy for Improvement of Drinking Water Quality
d) Monitoring and Control of Non-Revenue Water (NRW)
e) Effective Drinking Water Quality Monitoring and Surveillance
f) Creation of database including Maps
g) Achieving Service Level Benchmarks (SLBs)

4. ADDRESSING THE MAJOR TECHNICAL CHALLENGES

To ensure a sustainable and resilient urban water supply sector, transformative changes are required to address the above challenges. This Manual presents a detailed analysis of the major challenges faced by the Indian urban water supply sector and outlines strategies to achieve a successful transformation.

a) Water security including Quantity and Quality

Urban Water Security is defined as the dynamic capacity of the water system and water stakeholders to safeguard sustainable and equitable access to adequate quantities and acceptable quality of water, that is continuously, physically, and legally available at an affordable cost for sustaining livelihoods, human well-being, and socio-economic development. Water Security also ensures protection against water-borne contamination, water-related disasters, and for preserving ecosystems. The Manual covers all the aspects of planning and design to make the system water secure from the source to distribution.

Success of the water supply scheme directly depends upon the potential and perennial water source, which must be 95% reliable and dependable. Selecting sustainable sources and developing the same is very crucial for any water supply system. The surface and ground water sources are to be identified, studied with respect to quantity and quality, and analysed for the suitability. Once the suitability is established, various alternative ways to identify the location of source shall be studied and final selection is made. Further various alternatives/methods of treatment for different qualities of raw water are explained.

The planning in the water supply sector aims at creating schemes with holistic/ comprehensive approach that helps in effective water resource planning through Integrated Urban Water Resource Management (IUWRM) which is a subset of Integrated Water Resource Management (IWRM). IUWRM emphasizes the need for city water balance and water conservation through rainwater harvesting, use of recycled water along with conjunctive use of surface and ground water sources. Effective IUWRM helps to achieve the goal of converting existing intermittent water supply to continuous 24x7 PWSS and covering uncovered areas to ensure adequate water source to every household.

b) Conversion of Intermittent water supply to 24x7 PWSS with Operational Zones (OZs) and District Metered Areas (DMAs)

In many urban areas, including cities and towns, the water supply is characterized by interruptions, where water is available for only a limited duration each day, typically ranging from 2 to 6 hours. This intermittent water supply system poses significant challenges. During the periods when water is not flowing through the pipelines, there is a risk of contamination of the drinking water within the distribution network. Additionally, this intermittent supply contributes to high levels of Non-Revenue Water (NRW), which essentially means water that is lost or unaccounted for, leading to an uneven and inequitable distribution of water resources among residents. To address these issues, a shift from traditional centralized planning to a decentralized approach is necessary, incorporating the concepts of OZ (Operational Zone) and DMAs (District Metered Areas). The Bureau of Indian Standards (BIS) code IS 17482:2020 provides the guidelines and standards for planning and designing OZs and DMAs, offering a structured framework for improving water supply management in urban areas. This transition is critical for ensuring reliable and equitable access to clean drinking water for all residents.

The Chapter 2 of Part A of this manual provides the detailed procedure for gradual conversion of intermittent system to 24x7 PWSS with drink from tap. This includes a procedure for determining optimum boundary of operational zone, establishing DMAs which are hydraulically discrete. The OZ and DMAs can be suitably planned using Geographic Information System (GIS) based hydraulic modelling. The Ministry (MoHUA) has published advisory on "GIS mapping of water supply and sewerage infrastructures," which may be followed by referring the Ministry's web site.

The size of OZ should not be more than 50,000 population (ultimate) or 10,000 connections for plain areas and for hilly areas, maximum ultimate population per OZ should be 30,000 or 6,000 connections. OZs are further divided into sub zones called as *District Metered Areas* (DMAs). The range of connections in DMA shall be 500-3000 connections in plain areas and 300-1500 in hilly areas. However, in saturated/ high density population areas where land is a constraint, the norm of 50,000 population per operational zone may be relaxed in plain areas and ultimate population up to

75,000 to 100,000 shall be considered in operational zone with proper justification. However, the number of DMAs may be suitably increased by restricting a maximum of 3000 household connections per DMA.

DMAs are progressively chosen for providing 100% consumer metering and with bulk meter at entry of each DMA. Leakages in chosen DMAs are identified, quantified, repaired and arrested. The leakages in all the DMAs should be stopped to enhance adequate water supply in 24x7 pressurised water supply systems.

As most of the ULBs have designed their distribution system for 7 or 12m residual heads, in the past increasing the residual pressure to 17-21m is a challenge. One should not abruptly increase the residual pressure to 17-21m, but the pressure should be gradually increased in order to avoid sudden increase of leakages in the distribution system. In most of the cities elevated service reservoirs (ESR)s have not enough staging heights, so VFD pumps at the outlet of ESRs are suggested to increase the residual head. The detailed procedure is given in the Manual. Cities are advised to constitute NRW cell for effective monitoring and control of NRW.

c) Strategy for Improvement of Drinking Water Quality

It is the responsibility of the ULB/ Water Boards/ PHED etc. to supply water with adequate quantity & required pressure and acceptable quality meeting drinking water supply standards IS 10500: 2012 at every household. Following strategy shall be adopted for improvement of drinking water quality:

i. **Contamination of drinking water in pipeline during non-supply hours:** It occurs during non-supply hours in intermittent water supply system when the pipeline is empty which attracts outside contaminants, thus contaminating water. The strategy is to provide continuous water supply with adequate pressure so that the entry of outside contaminants is prevented.
ii. **Contamination of drinking water in customer underground tanks:** Customers construct underground (UG) storage tanks due to interrupted supply prevalent in the distribution system. In the past, the UG tanks were constructed with brick masonry where the joints in bricks are porous. Thus, not only the water leaks but it also allows outside contaminants to enter. Therefore, consumer UG tanks should be discouraged for the buildings up to the three storeys. The strategy is to provide continuous potable water supply with residual pressures of 17-21 m in case of Class I and II cities and 12-15 m for other and then subsequently remove UG tanks gradually. However, for high rise buildings, waterproof UG RCC/HDPE tanks are recommended with annual cleaning using chlorine.
iii. **Contamination of water in pipelines through ferrule points of HSCs**: As per various studies conducted, about 70-80% leakages occur at ferrule points and they become the point of contamination. This manual emphasises use of standard quality ferrules and pipes in addition to employing the services of licensed skilled plumbers for giving HSCs.
iv. **Contamination of raw water sources due to discharge of untreated wastewater**: Progressively over the past, the wastewater (partially treated or untreated) has been discharged into the water bodies. As the wastewater effluent and the pesticides leached from the agricultural fields discharged into the water bodies contain various contaminants including micro-pollutants, Endocrine Disrupting Chemicals etc., it is important to adopt high degree of treatment to protect the public health. Therefore, the conventional water treatment processes are ineffective to treat such variety of contaminants. The degree of treatment for raw water sources may be required to be enhanced with a judicious combination of appropriate treatment technologies by making additional investments. The suggested line of treatment options for contaminated surface water sources have been provided in Chapter 8.

d) Monitoring and Control of Non-Revenue Water (NRW)

Inefficient distribution systems, and unauthorized connections result in high levels of NRW. NRW is defined as the water that has been produced and is "lost" (leaks, theft or metering inaccuracies) before it reaches the customer. High value of NRW means lot of water is lost because of which the water utility gets less income, and the system becomes unsustainable. City requires all out efforts to reduce NRW level less than 15% by forming NRW cell.

DMA based approach ensures that the NRW reduction is achieved by monitoring the DMA level flows, pressures and quality. Metering along with automation will make the NRW and leakage control measured effective as the data will be real-time and used by the O&M personnel.

e) Effective Drinking Water Quality Monitoring and Surveillance

Drinking water quality monitoring is achieved by periodic sampling and analysis of water constituents and conditions. It is necessary to know whether water is contaminated physically, chemically and biologically. Thus, making arrangements for water quality monitoring and surveillance is a challenge. The water quality has to be maintained to ensure that the people can drink from tap. The Indian Standards and the resources needed for establishing a state-of-the-art water testing laboratory for effective testing and monitoring has been discussed in detail in Chapter on Water Quality Testing and Laboratory Facilities.

f) Creation of database including Maps

There is apathy of creating maps of the water infrastructure. 24x7 PWSS with DFT needs information of the existing water system in the form of maps. Unfortunately, maps of existing pipelines laid below ground are not available in most of the Urban Local Bodies (ULBs). Hence, creation of database of water infrastructure and maps both in physical and digital form is essential for effective planning, design, implementation, operation & maintenance and management.

GIS based survey with consumer data will help in developing the Network Models. The conditional assessment data will help in designing the existing as well as proposed augmentation schemes to deliver 24x7 PWSS with DFT.

g) Achieving Service Level Benchmarks (SLBs)

The targeted Service Level Benchmarks (SLBs) for water supply were notified by MoHUA in the year 2008. While planning the water supply schemes, the ULBs shall carry out the survey of its city/town and find out the baseline parameters of the performance indicators. The gaps between the benchmarks and the baseline parameters shall be worked out and the DPR shall be prepared to bridge the gaps so that the benchmarks shall be attained. Some of the service level benchmarks such as 24x7 pressurised water supply, 100% metering, control of NRW, and quality of water supply shall be considered as Key Performance Indicators in the tender document. In addition to the above, residual nodal pressure shall be included in the tender document as KPI. All water supply projects should be implemented with the objective to achieve the aforesaid SLBs and monitor the same though out design period. It is to be noted that by conversion of intermittent water supply to 24x7 pressurized water supply (24x7 PWSS with DFT) most of the SLBs will be achieved.

The manual strongly recommends planning, design and implementation of water supply projects based on operational zones and DMAs. A multi-pronged, people-centric approach for conversion from existing intermittent water supply to 24x7 PWSS with DFT has been suggested in this manual.

This manual emphasises 100% household coverage with pipe water supply and metering with differential tariff based on volumetric consumptions for ensuring financial sustainability of the 24x7 pressurised water supply systems.

However, to ensure speedy implementation of 24x7 PWSS with DFT project, the city needs to prioritise the implementation of various project components in a phased manner. In this regard, it has been suggested that the cities should initially implement water distribution network in the project area or the whole city by considering OZs and DMAs with inlet and outlet arrangements (bulk flow meters, isolation valves, pressure valves, HSC up to boundary of the premises etc.) to facilitate better utilization of the capital investment available under time bound missions like AMRUT 2.0 or State Plan Funds. Immediately after the formation of OZs and DMAs, the cities shall initiate action to connect the house service connections with houses along with water meters for gradually achieving 24x7 PWSS in one after another DMA and upscale to project area or entire city in a phased manner as clubbing the laying of main distribution network and providing house service connection with meters simultaneously will delay the commissioning of the overall project.

5. COMPOSITION OF CHAPTERS

Part A of this manual comprises of 16 chapters and the brief details covered are as follows:

Chapter 1: Introduction provides the status of water supply in urban India, the issues and challenges in urban water supply sector, the demerits of intermittent water supply and the need for conversion of intermittent water supply to 24x7 PWSS with Drink from Tap facility and its merits, concept of decentralised Urban Water Supply System, Water Policies and Governance etc.

Chapter 2: **Planning, Investigations**, Design and Implementation provides guidelines on planning, design, implementation of 24x7 PWSS with DFT projects with Drink from Tap facility, gradual conversion of existing intermittent water supply to continuous pressurised water supply systems with retrofitting, norms for planning and design, concepts that should be adopted and the investigations that are necessary to be carried out for optimal planning and design with comprehensive management strategy. GIS based network modelling and the processes to be adopted are explained in detail. Various case studies of successful implementation of 24x7 PWSS with DFT are provided.

Chapter 3: Project Reports. This chapter explains all the documentation needed at various stages of the project development, viz. from inception, pre-feasibility, feasibility and detailed engineering design stage. The templates/checklist of the information to be proved in the reports is included. This chapter also includes environmental, social and gender safeguard components which are very crucial for the implementation of projects and for availing funding assistance from multi-lateral agencies.

Chapter 4: Planning & Development of Water Sources provides guidelines for Planning and Development of Water Sources, assessment of surface and ground water, development of surface and sub-surface sources, ground water recharge methodologies. The objectives of Integrated urban water resource management and the need for city water balance plan has been explained in detail.

Chapter 5: Pumping Stations and Pumping Machinery provides guidelines on Pumping Station and Machinery. Pumping design principles along with designs, selection of best machinery/combination for efficient selection is explained in detail. Criteria for selection of pumps, variable frequency drive pump, energy efficient motors, pumps based on class, motor rating, pumping station, etc. and other design considerations have been explained.

Chapter 6: Transmission of Water, provides guidelines on the design of the transmission main system which supplies water to various service reservoirs. The transmission main should be designed for equalization of pressure heads at the full supply level of each service tank. This ensures the equal distribution of water even in uneven terrain. The transmission system design along with sample design for economical selection of pipe diameter and material is discussed in this manual.

Chapter 7: Water Quality Testing and Laboratory Facilities provides a comprehensive guideline on Water Quality Testing and Laboratory Facilities to maintain and monitor the water quality of sources as well as drinking water surveillance in water supply distribution network. The Indian Standards and the resources needed for establishing a state-of-the-art water testing laboratory have been explained. The frequency of supply is discussed in the chapter. The equipment, machinery, consumables, and manpower are thoroughly discussed.

Chapter 8: Conventional Water Treatment discusses various alternatives/methods of treatment process to be followed depending on the raw water quality and are explained with detailed design and examples for each component of the process chain including their advantages and disadvantages. It also presents computer aided optimal design of water aided system.

Chapter 9: Disinfection discusses disinfection methodologies and their benefits. The advantages and limitations of various disinfection methods, combinations of disinfections have been discussed.

Chapter 10: Specific Treatment Processes provides guidelines on specific treatments needed for sea water desalination, softening, removal of Arsenic, Iron, Manganese, Fluorides etc.

Chapter 11: Pipes and Pipe Appurtenances provides guidelines on various Pipes and Pipe Appurtenances. Laying, jointing, testing of pipelines, advantages and disadvantages for different pipe material have been explained. Different valves, manhole inspection and jointing have also been discussed.

Chapter 12: Service Reservoirs and Distribution System explains in detail the design of distribution system for OZs and DMAs, design and rehabilitation of existing distribution system and the service reservoirs with all concepts of network modelling, including network management and NRW reduction process by water estimating losses using water auditing process. It also describes various requirements and materials for providing HSCs and water meters.

Chapter 13: Water Meters provides guidelines on various types of water meters and flow meters with all the technical details and specifications. The installation, testing, calibration, repair and troubleshooting are also discussed.

Chapter 14: Automation of Water Supply Systems provides guidelines on various Automation instrumentation and systems used in various components of the water supply system, including Telemetry, SCADA, instrumentation, IoT, Digital Twin etc. The guidelines for controlling NRW in DMA are discussed with modern communication technologies.

Chapter 15: Water Efficient Plumbing Fixtures discusses the use of Efficient Plumbing fixtures for water conservation as per the Indian Standards 17650 (Part 1 and 2) have been explained.

Chapter 16: Planning and Design of Regional Water Supply System provides guidelines for planning, design and implementation of Regional Water Supply Schemes for Urban, Peri- Urban and Rural areas.

CHAPTERS

CHAPTER 1: INTRODUCTION

1.1 Background

Safe water in adequate quantities is essential to all forms of life on earth. It is the backbone of a healthy economy and greatly contributes to poverty removal. Safe drinking water should be reliable, accessible, and accepted for all the users.

The United Nations (UN) declared access to safe drinking water as a fundamental human right. The UN further stated that drinking water is an essential step towards improving living standards. The UN also declared Millennium Development Goals (MDGs) and the Sustainable Development Goals (SDGs) with the goal of access to water. The SDG's goal 6 states that "Water sustains life, but safe clean drinking water defines civilisation."

National Institution for Transforming India (NITI) Aayog (2019) stated that "India is a home to about 17% of world's population but has 4% of the world's freshwater resources." Every year India gets 4,000 billion cubic metre (BCM) water as annual renewable water resources. India is placed 9th in the hierarchy of annual renewable water resource. It receives an average annual precipitation in the range of 750-1,500 mm. From the surface water and replenishable groundwater, 1,869 BCM water is available but only 60 % of it can be beneficially used. Thus, India's total available water resource is 1,122 BCM out of which 690 BCM is surface water and 432 BCM is in the form of ground water. The surface and ground water approximately contribute 61% and 39% of total availability.

Though abundant water is available, the country has great variation of time and space when it comes to rainfall. When the northeast rivers flow in high discharge, rivers in the southern part of India carry low discharge. In India, 90% of flow occurs in the four months of monsoon, and 50% of this occurs in just 15 rainy days.

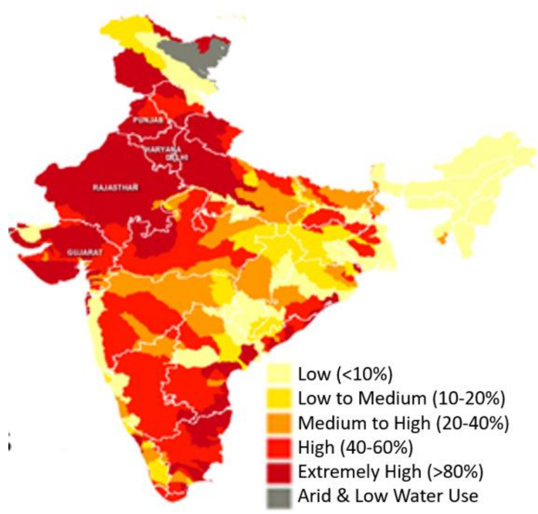


Figure 1.1: Water stress in India (Source: World Resources Institute)

NITI Aayog (2019) stated that nearly 60 crore people (Figure 1.1) living in India face high to extreme water crisis. It further mentioned that about 40% Indians will have no access to drinking water by 2030. The report finds that the "Water Gap" can be closed by undertaking measures such as boosting water use efficiency and lessening the water intensity of the economy by demand management and good measurement practices. The annual per capita availability of water is expected to reach 1,341 cubic metre per capita per year in 2025 to 1,140 cubic metre per capita per year in the year 2050 thus leading to severe water stress.

NITI Aayog also estimated that about two lakh Indian persons die yearly due to inadequate and unsafe drinking water. In India, huge quantity of wastewater is generated. Mismanagement of wastewater and that of liquid waste causes contamination of ground water, and poor sanitation conditions. Besides this, poor hygiene habits cause waterborne diseases among the large portion of population, especially among the poor.

1.2 History of Urban Water Supply

Just after independence, water supply in Indian cities was not satisfactory. Only 16 % of the total number of towns in India (Environmental Hygiene Committee, 1949) had protected water supplies which served 6.15 %of total population or 48.5 %of the urban population. Water was supplied at 2 to 40 gallons (10 litres to 180 litres) per capita per day. Only a few waterworks were augmented by 1949. Among these were water supplies of Delhi, Bombay, Kanpur, and Bangalore. In most places, new schemes were shelved. However, situation improved since then.

1.3 Present scenario of urban water supply

NITI Aayog (2019) also stated that 93% of India's urban population had access to basic water supply. Distribution of households according to the primary source of drinking water as reported by Census 2011 is shown in Figure 1.2. It can be seen that 62% of households have access to treated tap water. This means nearly 38%of urban households have no access to treated tap water. They have to depend on other sources of water. As per Census 2011, urban population was 31.16% and 370 million were inhabiting urban India out of which 65.4 million were slum dwellers.

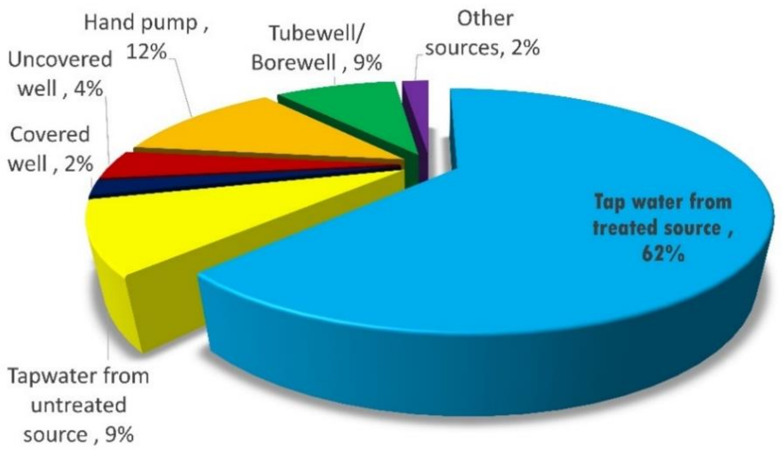


Figure 1.2: Distribution of Households according to Source of Water

Source: Analysis of Census 2011 Data

Present challenge is to provide treated water to the 38% households which are without access to treated tap water. The urban population is expected to grow to 590 million by the year 2030. Thus, there is a great challenge ahead to supply every household by treated water tap.

Universal piped water supply coverage was the objective under the Atal Mission for Rejuvenation and Urban Transformation (AMRUT) in 500 cities of India. The mission was launched by the Ministry of Housing and Urban Affairs (MoHUA). As of November 2023, 1.73 Crore new tap connections have been provided under AMRUT. AMRUT 2.0 was launched by MoHUA in October 2021 with an objective to provide water security and 100% functional tap connections in all cities and towns in the country with the target of 2.68 Crore connections till 2026.

One of the objectives of the AMRUT 2.0 is to provide 24x7 pressurized water supply system (24x7 PWSS with DFT) with the drink from tap facility in at least 1 zone or 2000 connections in 500 AMRUT cities. There lies a great challenge ahead - to supply continuous water supply to every household with functional water tap.

1.4 Major Challenges in urban water supply

1.4.1 General Challenges

Despite the advancements in water sector, access to piped water supply in urban areas is not yet universal. Thus, there lies a great challenge ahead to supply potable water to every household with functional water tap.

Waterborne diseases is one of the reasons that the infant mortality ratio of India is on higher side, which is 26.7 deaths per 1000 live births in 2022. Thus, low-income group people have to make expenses on health aspect. The economic burden due to this is about USD 600 million (Rs 4,920 crores) per year in India. The waterborne diseases are rampant in drought- and flood-prone areas, which affected a third of India's population in the past couple of years.

Another challenge is the extreme ground water depletion rate (https://www.unicef.org/india) in two-thirds of India's 718 districts. Due to rapid increase in the drilling operations since the last two decades, India became the largest user of ground water. Joint Monitoring Programme (JMP) of water supply, sanitation and hygiene of WHO/UNICEF in 2017 stated that through about 30 million access points in India, groundwater supplies drinking water to 85% in rural areas and 48% of water requirements in urban areas.

Besides above, there is another important challenge of supplying pressurised 24×7 continuous water supply to all the people residing in urban areas. 24×7 pressurised water supply system needs information of the existing water infrastructure in the form of maps and database. Unfortunately, maps and databases of existing pipelines laid below ground are not available in most of the urban local bodies (ULBs). Hence, creation of such maps in GIS format is the big task and challenge.

Availability of continuous electricity is important for running the pumps. In many cities/towns, due to daily tripping, there is breakdown of electricity. As a result, during this period, pipeline becomes empty and requires more time to refill with water. This creates pressure-deficient conditions. Hence, providing continuous electricity is a challenge. This manual recommended to use express feeder with bypass arrangement to solve this problem.

Engineering and Technical Challenges

There are several technical challenges which are enumerated as follows:

(i) Highly contaminated raw water sources
Raw water means the water we get from rainwater, groundwater, surface water, well water, lakes, rivers, etc. One of the major environmental issues in India is that of water pollution. Harmful germs find entry through untreated sewage - the largest source of pollution. Other sources include agricultural runoff and unregulated small-scale industry such as fertilisers, pesticides, industries, sewer overflows, and storm water.

(ii) Improper planning and design of water supply network

It leads to the following problems:

a) Shortage of water: If the source is inadequate and undependable (<95% reliable), the shortage of water will be experienced. If the supply of water is restricted, then pressure deficiency in the nodal pressures will be formed.
b) Haphazard laying: It is normally observed that as and when existing pipeline cannot cope up high demand, parallel pipelines in the covered areas and extension of pipelines in

uncovered areas are provided by ULBs. This gives rise to clumsy network causing inequitable distribution and insufficiency of pressures, thus making the system uncontrollable from O&M point of view.

c) Cross connections: The ULB's operating staff generally tends to find temporary solutions to the supply problems and opt for cross connecting distribution network pipelines, without any scientific assessment/study, and on ad hoc basis. No records are generally maintained of these cross connections in most of the cities and towns.
d) Adding of dwarf and small capacity ESRs: In many cities, ULB chose a way of adding small capacity and dwarf service tanks. As staging height of these reservoirs is less, it is obvious that the norm of minimum residual pressure cannot be achieved from these reservoirs of low staging height.
e) Exceptionally big zone: In many cities, excessively big operational zones (OZs) are provided with a single service tank to serve large population. This causes dropping of pressures and the system is compelled to operate on intermittent system, resulting into contamination of water in the pipeline during non-supply hours, and high non-revenue water (NRW) leading to in-equal water supply.
f) Low nodal pressures: In some of the cities, the distribution system has been designed for low residual nodal pressure due to which many parts of the city are not getting water with adequate quality and pressure.
g) A large number of consumers' underground (UG) tanks: In most of the cities, consumers have their UG storage tanks. These UG storage tanks leak and also allow outside contaminants to enter in. Due to unbalanced capacity, high-income residents are using more water and low-income group are starving for water.
h) Inequitable flow and pressure: The distribution system is laid on high altitude and low-lying areas of the city. Residents in the low-lying areas get excess water and high areas get less with low pressure.

(iii) Intermittent water supply leading to contamination of drinking water during non-supply hours and formation of THM after post chlorination.
Contamination of drinking water in pipeline occurs during non-supply hours. In intermittent water supply system, it occurs during non-supply hours when pipelines are empty.

(iv) High NRW and inequitable Water Supply:
Generally, NRW is observed in range of 30%-50%. NRW is the water that has been produced and is "lost" before it reaches the customer. In real loss, water is lost due to physical leaks and in commercial loss, it is due to theft or metering inaccuracies.

As the water in the system is loaded with energy, high value of NRW indicates that energy is poorly managed which is lost.
High value of NRW means lot of water is lost because of which the water utility gets less income and the system becomes unsustainable.

(v) Lack of monitoring of drinking water quality and NRW using smart technologies
Water quality monitoring is achieved by the sampling and analysis of water constituents and conditions. It is necessary to know whether water contains pollutants and also pesticides, metals, and oil.
In the absence of water quality monitoring:

- it is difficult to identify whether waters are meeting designated uses;
- it is difficult to identify specific pollutants and sources of pollution;
- it is difficult to determine trends over time;

- early warning "screen" of potential pollution problems is not available.

1.4.2 Challenges in O&M of Water Supply System

The operation and maintenance (O&M) of water supply systems present several challenges, ranging from technical issues to financial constraints. Some of these are as follows:

1. ageing infrastructure, which requires constant repairs and maintenance to ensure it functions efficiently and effectively.
2. availability of skilled personnel to operate and maintain the complex water supply systems.
3. financial sustainability.
4. control of NRW.
5. lack of metering to ensure sustainability of O&M.

Addressing these challenges requires a comprehensive approach that involves effective planning, adequate investment, and a skilled workforce, along with the adoption of sustainable practices to ensure long-term climate-resilience of the water supply system.

1.4.3 Management & Financial Challenges

Effective water supply management in Indian urban areas is hindered by several managerial challenges that impact the planning, operation, and maintenance of water systems. Addressing these challenges is crucial to ensure the efficient and sustainable provision of clean water to growing urban populations. These challenges are multifaceted and require comprehensive reforms to address them adequately which are:

1) Fragmented governance structure: The responsibility for urban water supply management is often fragmented among various government departments and agencies at different levels, including municipal corporations, state water boards, and state governments. Lack of co-ordination and clear division of roles can lead to inefficiencies and overlapping responsibilities.
2) Outdated legal framework: Many Indian states have outdated and inadequate water laws and regulations that do not align with the current urban water supply challenges. Reforms are needed to develop comprehensive water laws that address emerging issues and support sustainable water management practices.
3) Limited accountability and transparency: Transparency and accountability in the urban water sector are often lacking, making it difficult for citizens to understand water service provision, tariff structures, and investment decisions. Improved transparency and accountability mechanisms are necessary to build public trust and ensure efficient resource allocation.
4) Financial viability of utilities: Many urban water utilities face financial challenges due to high NRW, low tariff collections, and inadequate cost recovery. Ensuring the financial sustainability of water utilities is essential to maintain and upgrade infrastructure and provide reliable services.
5) Limited Community Participation: Meaningful community participation in decision-making processes related to water supply management is often lacking. Engaging communities can lead to better understanding of local needs and concerns and foster a sense of ownership over water resources.
6) Inadequate Capacity and Skills: A shortage of skilled professionals and technical expertise in urban water management poses challenges in planning, operation, and maintenance of water supply systems. Building institutional capacity and investing in workforce development are crucial to improve overall water governance.

7) Public-Private Partnerships (PPPs): The implementation of PPP models in the water sector has been met with mixed results in India. Balancing private sector efficiency with public interest, equitable access, and affordability remains a challenge.
8) Inadequate regulation and enforcement: Regulation of the water sector is often weak, leading to non-compliance, unauthorised connections, and illegal water use. Effective regulation and enforcement mechanisms are essential to ensure adherence to standards and promote responsible water use.
9) Inadequate infrastructure planning and asset management:
   a. Lack of comprehensive infrastructure planning and asset management leads to suboptimal investments, inefficient resource allocation, and difficulties in maintaining and upgrading water infrastructure.
   b. Data collection and management information systems:
   c. Accurate data collection, analysis, and management are essential for informed decision-making. However, many water utilities lack robust data collection systems and data-driven management information systems and reporting practices.
10) Lack of integrated urban water resource management (IUWRM): There is a disconnect between water supply, wastewater management, storm water management, and groundwater management. The absence of an integrated approach hinders sustainable water resource management and creates challenges in addressing water quality and availability issues holistically.
11) Climate change and resilience: Climate change impacts on water availability and extreme weather events pose significant challenges to urban water supply management. Building climate resilience and incorporating climate adaptation measures in water planning are crucial.
12) To address these challenges, comprehensive reforms are needed, including revising legal frameworks, strengthening institutions, improving co-ordination between stakeholders, promoting community engagement, and investing in modern technology and infrastructure. A holistic and integrated approach to urban water supply management is essential to ensure sustainable water services and meet the growing demands of India's urban population.

1.5 Disadvantages of Intermittent Water Supply

Intermittent water supply has several disadvantages. Its comparison with 24×7 pressurised water supply system is shown in Table 1.1.

Table 1.1: Comparison of Intermittent water supply with 24×7 pressurised water supply system

SN	Demerits of Intermittent System	Merits of 24×7 System
1	Large doses of chlorine	Reduces contamination
2	Capacities underutilised	Better health outcomes
3	Valves - wear and tear	Life of network increases
4	More manpower - Zoning	Reduces contamination
5	Large sizes of pipes	Better demand management
6	Supply hours affect poor	Reduces consumption
7	Storage is required	Consumer satisfaction
8	Pay for pumping	Willingness to pay-slums
9	High health risks	Time is managed effectively
10	Meters go out of order	Time for rewarding activities
11	Store and throw water	Lowers health risks

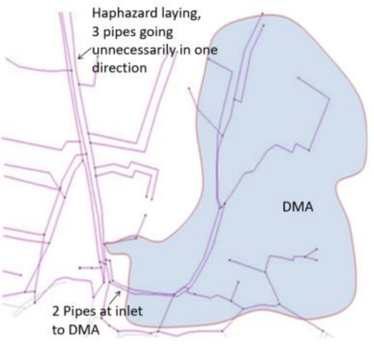
SN	Demerits of Intermittent System	Merits of 24×7 System
12	Wastage of treated water	Attracts industries
13	Water is not easily available to low-income people	Water is supplied to all including low-income people

In an intermittent water supply system, water is supplied only for few hours in a day which causes great inconvenience to consumers, as time of supply does not suit to them. Consumers tend to keep taps open during the no-supply period and this results in wastage of water when the supply starts.

1.5.1 Reasons of Intermittent Water Supply

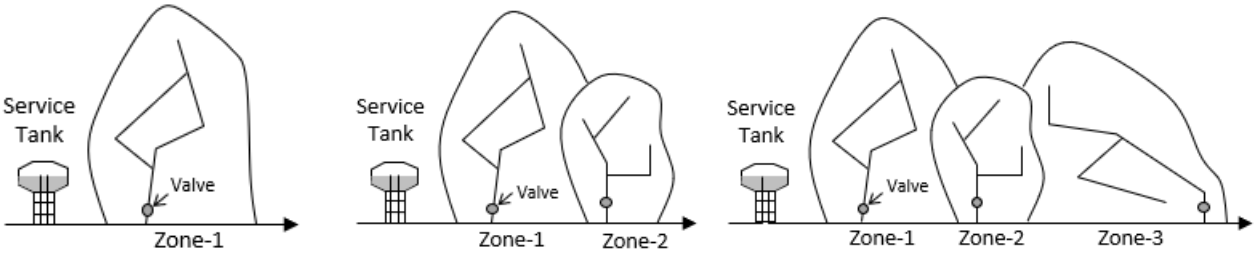
(a)Haphazard Laying: It is normally observed that as and when the demand of water increases, pipelines are laid haphazardly by ULBs in expanding areas. One such incident in one city is shown in Figure 1.3(a). This gives rise to clumsy network causing inequitable distribution and insufficiency of pressures, thus making system uncontrollable from O&M point of view.

In the initial period after commissioning of the scheme, the system operates satisfactorily as shown in Figure 1.3(b).



- ULB laid pipelines haphazardly,
- More than one pipe going in the same direction and same locality.
- No control on distribution
- Due to multiple lines, nodal pressures drop.
- Due to two inlets to DMA, it requires two isolation valve, Bulk meters, PRV and FCV

(a): Haphazard laying of pipes



(b): Design Stage (c): After few year (d): After few more years

Figure 1.3: Zoning of water distribution system practised.

Later, since ULB added pipelines erratically without proper design check, hydraulics gets vitiated, and pressures drop. So, after few years when demand increases, another zone is required to be added as shown in Figure 1.3(c). Again, after few more year's additional area, a third zone is added as shown in Figure 1.3(d). Subsequently all this finally compels transformation of present system into an intermittent system along with additional transmission lines and storages tanks.

(b)Adding of Dwarf and Small Capacity ESRs: In one city, after commissioning of its distribution system around in 1990, ULB added 21 ESRs of only 8 to 10 m staging height and a small capacity of 25,000 to 50,000 litres capacity. As staging height of these tanks was less, it is obvious that the norm of minimum residual nodal pressure cannot be achieved because of these tanks of low staging height. This is a common scenario observed in many cities.

(c)Huge Service Area: In one city, three service tanks have a common huge service area supplying water to 79,790 population. This service area is supplying water for one hour to near area adjacent to the ESRs, but people at farthest boundary of this service area get water supply for just 30 minutes.

(d)*Other major reasons:* for intermittent supply are as follows:

- non-availability of continuous electric supply;
- continuation of water distribution system (WDS) beyond its design life;
- non-availability of adequate quantity of water at source;
- unexpected or unbalanced growth during design period;
- heavy leakage losses;
- improper layout;
- unmetered supply;
- improper planning and design of network and poor O&M.

1.5.2 Sustainability of Water Sources

Water source is the soul of any water supply project. The source should be such that it will supply water incessantly for all the seasons. City water supply requires 95% dependable source. Sustainability of the water source is achieved by adopting IUWRM which is defined as a technique that encourages co-ordinated land and water development and management in order to maximise economic and social welfare in an equitable manner and is needed for comprehensive planning of river sub-basin and groundwater sources. Details of IWRM and IUWRM including City Water Balance Plan is discussed in section 4.13 and 4.14 of Part A Manual.

1.5.3 Necessity of Shifting from Intermittent to 24×7 Water Supply

Urban water sector is facing the challenges of poor quality of water. Intermittent water supply often results in contaminated drinking water and is one of the reasons of considerable mortality ratio of 27.6 in year 2022 in India. Mechanism of water contamination is shown in Figure 1.4. During non-supply hours, there is a vacuum inside pipelines due to which outside dirt/contaminants find entry into the pipelines, thus, water is contaminated. When supply of water starts, the contaminants are mixed with the treated water, thus, contamination takes place.

Unlike in intermittent supply, in 24×7 water supply system, by definition, pipelines are pressurised and hence outside dirt cannot find entry (Figure 1.5) inside, hence water retains its quality. Because of contamination in supply, people tend to purchase small reverse osmosis (RO) machines in their homes. Thus, coping costs such as developing storage facility, pumping water to roof-level storage, household treatment facility and their maintenance are on the rise (Amit and Sasidharan 2019) about Rs 558 to 658 per month for piped and non-piped households respectively. In addition to this, power is required and about two-third water from RO is wasted as reject water.

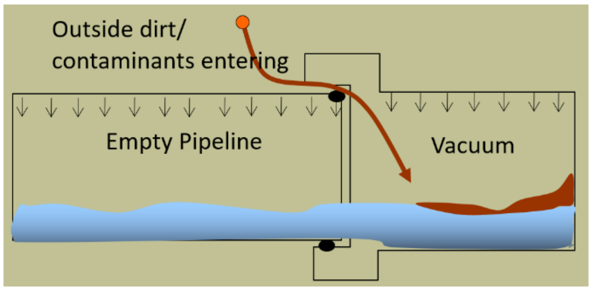


Figure 1.4: Intermittent System

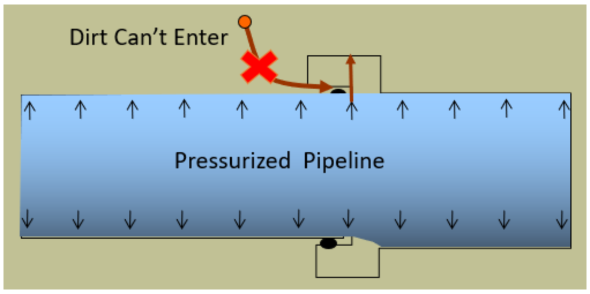


Figure 1.5: 24×7 Pressurised Water Supply System

In developed countries, water is provided on a 24×7 basis. Some of the countries in Africa also provide 24×7 pressurised water supply system. Intermittent water supply system is practised only in South-Asian countries like India. Hence, it is the most important challenge ahead of India to convert its intermittent water supply to 24×7 system.

1.6 Sector Organisation

1.6.1 Government of India (GoI)

In India water is a state subject, but the provisions are quite complicated. The primary entry in the Constitution relating to water is at 17 in the State list. It brings water including water supplies, irrigation and canals, drainage and embankments, water storage and waterpower under state list.

Though water is in the State list, there was a need to have a centralised organisation to guide the state's water supply projects. Therefore, the Environmental Hygiene Committee, in its report in 1949, recommended to form a centralised agency of Central Public Health and Environmental Engineering Organisation (CPHEEO).

(i) CPHEEO

CPHEEO has been in existence for more than 67 years since its raising under the Ministry of Health in 1954. It has participated in all important sanitation programmes for the country. CPHEEO has been affiliated to the Ministry of Housing and Urban (MoHUA).

The organisation not only supports the Ministry in policy formulation but also handholds States by way of technical advice, guidelines, scrutiny, and appraisal of schemes and propagation of new technologies in the field of water supply and sanitation including municipal solid waste management. CPHEEO deals with the matters related to urban water supply and sanitation including solid waste management in the country. CPHEEO plays a vital role in processing the schemes posed for Bilateral and Multilateral funding agencies such as World Bank/JICA/ADB/KFW/AFD and other external fund agencies.

(ii) Formation of Jal Shakti Ministry

GoI formed Jal Shakti Ministry in 2019 by merging two ministries - Ministry of Water Resources, River Development and Ganga Rejuvenation and Ministry of Drinking Water and Sanitation (Rural).

(iii) AMRUT

AMRUT was launched in June 2015. In 2019, AMRUT 2.0 was established. Some of the salient features of AMRUT 2.0 are as below:

- total outlay of Rs. 299,000 Cr.;

- securing tap and sewer connection of estimated 2.67 crores urban connections;
- 500 AMRUT cities are mandated to implement 24×7 water supply project in at least one ward, or one district metered area (DMA) with 2000 household;
- reform incentives and additional funding extended for performance based 24×7 water supply projects;
- water balance and NRW reduction to 20% is mandated;
- outcome based financing and innovative contract structure like PPP, HAM, etc.

1.6.2 State Governments

As water is a State subject, the States have set up water-related departments such as Water Resource Department, State Water Supply Boards, Zilla Parishads, etc. These departments prepare water supply schemes for urban and rural sector of the states.

1.6.3 Urban Local Body (ULB)

India Infrastructure Report (2011) further states that "the 74th Amendment to the Constitution of India recognises local self-governance as an enforceable ideal and helps the state governments to constitute ULBs. The 74th Amendment also requires that the Legislature of a State may, by law, endow the Municipalities with such powers and authority as may be necessary to enable them to function as institutions of self-government." Thus, the issues that may be entrusted to the Municipalities include water supply for domestic, industrial, and commercial purposes. With such mandate, ULBs started executing water supply schemes with financial assistance from the State as well as from the Central Government.

1.7 Initiatives of GoI

Service Level Benchmarking

Considering importance, the Ministry of Housing & Urban Affairs (MoHUA), GoI, has launched the Service Level Benchmarking (SLB). As part of the ongoing endeavour to facilitate critical reforms in the urban sector, the MoHUA has adopted national benchmarks in four key sectors of water supply, wastewater, solid waste management (SWM), and storm water drainage. There is, therefore, a need for a shift in focus towards service delivery. These service level benchmarks have been developed for assessing performance of ULBs in providing water supply services. Such performance indicators, targeted benchmarks, and baseline performance figures are shown in Table 1.2.

Table 1.2: Performance indicator and benchmark for water supply services

S. N.	Performance indicator	Targeted Benchmark	Average values in India
1	Coverage of water supply connections	100%	70%
2	Per capita supply of water (LPCD)	135*	114
3	Extent of metering of water connections	100%	22%
4	Extent of NRW	15%	31%
5	Continuity of water supply	24 hours	2.7 hours
6	Quality of water supplied	100%	95%
7	Efficiency in redressal of customer complaints	80%	89%
8	Cost recovery in water supply services	100%	72%
9	Efficiency in collection of water supply-related charges	90%	60%

Source: PAS-SLB data from www.pas.org.in covering 900 cities in five States

For cities having population more than 10 lakhs, the target benchmark is 150 LPCD. The breakup of water requirement (IS 1172:1993), is shown in Table 1.3.

Table 1.3: Average water use per person per day in urban area

SN	Purpose	Quantity (LPCD)
1	Drinking	5
2	Cooking	5
3	Bathing	50
4	Toilet flushing	30
5	Washing utensils	15
6	Washing the house	10
7	Washing of clothes	20
	Total	**135**

1.8 Emerging Trends and Technologies

1.8.1 Climate Change

Climate change alters hydraulic cycle and has considerable impact on water. It changes the timing and intensity of the rainfall. Monsoon vagaries has impacts on water supply and sanitation of many cities whose population and demand of drinking water is ever increasing. It directly affects the quantity and quality of water resources.

In India, it is believed that impacts of climate change on water supply and sanitation may affect the achievement of the MDGs and that of SDG number 6.

1.8.2 Impact of Climate Change on Piped Water Supply:

Piped water supply system of city is vulnerable to extreme rainfall events. On 26th of July, 2005, Mumbai Metropolitan Area (having 20 ULBs) had witnessed such extreme rainfall (955mm in 24 hours). It had affected all the 20 water supply systems in the region. Subsequently, the Government of Maharashtra created interlink grid joining different sources of water supply as resilient measure. This arrangement is working efficiently.

Heavy rainfall events increase loads of suspended solids (turbidity) in reservoirs that are built as source of water supply. Increased turbidity increases load on water treatment plant consuming more coagulant doses and requires increased doses of chlorine disinfectants.

1.8.3 Response to Droughts

Many cities have to curtail water supply in the event of low rainfall. In such situations, pressure-deficient conditions are formed affecting service delivery. City administrations have to rationalise water distribution. For avoiding such situation, water needs to be reserved in the dams.

1.8.4 Integrated Urban Water Resources Management (IUWRM)

IUWRM is a participatory planning and implementation process. It is based on scientific approach in which the stakeholders decide how to meet society's long-term needs for water while maintaining essential ecological services and economic benefits.

The main elements of an IURWM system are:

- supply optimisation;
- demand management including cost-recovery policies;
- equitable access to water resources through participatory and transparent management;
- improved policy, regulatory and institutional frameworks;
- inter-sectoral approach to decision-making, combining authority with responsibility for managing the water resource.

1.9 Revision of Manual

Way back in 1949, the report of Environmental Hygiene Committee was accepted by the GoI, which stated, "Intermittent water supplies should be discouraged as far as possible. It results only in dissatisfaction, waste of water, inequitable distribution, and risk of contamination of water by back siphonage or in suction during hours of low pressure. Intermittent supplies are also open to the objection that the flushing of closets is interrupted, and the fighting of fires is impossible during the hours of interruption. It has been demonstrated at Lucknow that the water-works authorities can successfully supply water all the 24 hours, educate a community used only to intermittent supply to adapt themselves to continuous supply and reduce consumption."

This recommendation shows the long-lasting aim of improving service delivery of water supply to provide pressurised continuous water on 24×7 basis. Even though the present progress in that direction is not tangible, it is a time to work to achieve above goal ultimately throughout the country. The AMRUT 2.0 programme envisaged to provide 24×7 pressurised water supply system with drink from tap facility, GIS based master plans of towns and target for reduction of NRW to 20%. AMRUT 2.0 programme envisioned incentive-based reforms planning and implementation of projects in PPP mode in water sector, especially in cities with population below ten lakhs. All the urban water supply schemes are designed and operated as per the current CPHEEO (1999) norms and Service Level Benchmarks (SLBs).

1.9.1 24×7 Pressurised Water supply

Though the current manual (1999) recommends continuous 24×7 pressurised water supply system with minimum peak factor, important topics such as methodology of OZs, DMAs, water loss reduction programme, which are the essential building blocks of 24×7 system are not mentioned. If the OZ is not sized, designed, and maintained properly, it leads to malfunctioning of storage reservoirs like emptying and overflowing. Moreover, if the DMAs are not properly created (hydraulically discrete and with 100% consumer metering), it is not possible to compute level of NRW which is required as first step in the programme of water losses reduction. All these require Decentralised Planning.

1.9.2 The Concept of Decentralised Urban Water Supply System

Decentralised planning system solves the complex problem by breaking it into smaller sub problems (Figure 1.6) which are then initially solved. Finally, by combining the solutions of small problems together, the original complex problem can be resolved.

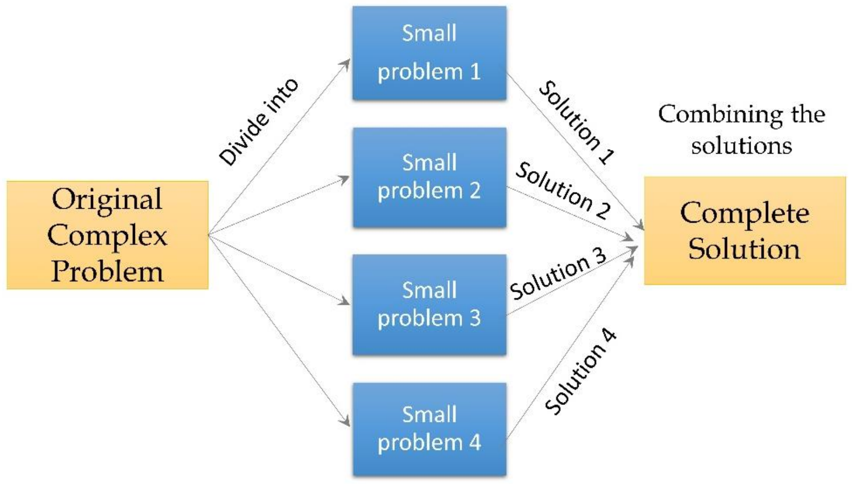


Figure 1.6: Principle of Decentralised Planning

Keeping this principle in mind, and considering the best practices adopted in the developed countries, thrust is given in this manual to the concepts of OZs. Converting water supply in each of them finally helps to switch city's intermittent water supply to 24×7 water system. For this purpose, a city is divided into manageable zones called OZs (Figure 1.7) which are further divided into subzones called as DMAs.

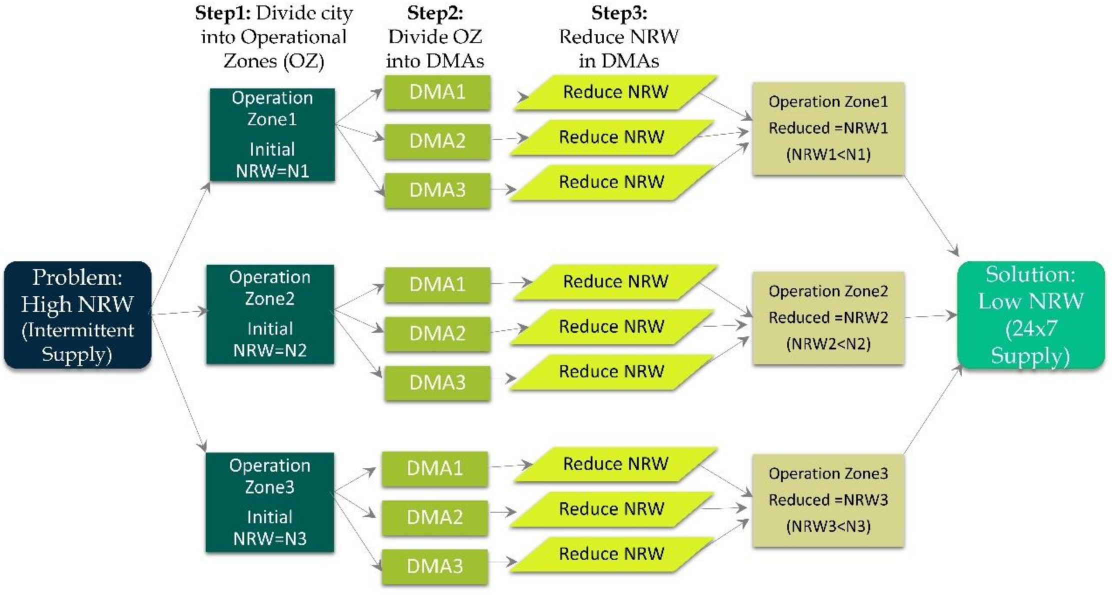


Figure 1.7: Application of Decentralised Planning in Water Supply

DMAs are progressively chosen for providing 100% consumer metering and with bulk meter at entry of DMA. Leakages in chosen DMAs are identified, gets quantified, and are removed. The leakages in all the DMAs should be stopped, and water, that otherwise would be lost, is saved which helps in increasing hours of supply. This is the basic principle of converting intermittent systems in to 24×7 systems. Each individual DMA is tackled in this way and their combined success in increasing water supply duration finally converts intermittent system of city to 24×7 water system.

1.10 Uniqueness of this Manual

This manual provides the detailed procedure for conversion of intermittent system to pressurised continuous 24×7 system. This includes a procedure for determining optimum boundary of OZ, establishing DMAs with various tests required for making it hydraulic discrete, comprehensive design of transmission main, rational design capacity of service tanks for 24×7 system, retrofitting and rehabilitation of water distribution networks, proper material selection, and control valves for 24×7 system.

It is well known fact that there are several problems in supplying water through distribution system giving rise to inequitable distribution, lack of pressures in higher elevation areas, high rate of NRW and problems related to quality of water. Most of the cities have clumsy and complicated distribution system. Because such situations were in existence before advent of DMAs, leakages were tackled in a passive way, i.e., leaks were repaired only when they were visible.

All the above mentioned problems can be solved by scientifically designing OZs and DMAs so that the main problems of high NRW and inequitable distribution can be effectively solved in decentralised manner. The demand management is most important. For demand management, 100% consumer metering with telescopic rate of tariff is required which helps in computation of NRW and subsequent water loss reduction. Elimination of illegal connections and volumetric telescopic tariff will further save water. The saved water is used for extending supply hours and finally converting the scheme in to 24×7.

Geographic Information System (GIS) and network technology for hydraulic modelling are also discussed in this manual. Hydraulic model, which simulates entire distribution pipe network, has been discussed at length in this manual. Using the planning tool of GIS, the methodology such as forecasting ward-wise population and demand allocation using forecasted population density have been discussed. Apart from this, the scientific art of making equitable distribution of water has been discussed. Thus, this Manual helps to improve service delivery of water supply system and would help to finally transform existing water supply systems into a 24×7 system.

The following missing new design procedures are discussed in this Manual:

1) Design of OZs and DMAs is included in this Manual. Distinctiveness of the present decentralised approach is to consider one OZ for each service reservoir. This is achieved by grouping the reservoirs as per characteristics of terrain which becomes easily possible by use of powerful GIS tool.
2) If the OZ is not sized properly, it leads to malfunctioning of reservoirs like *emptyin*g and *overflowing*.
3) There are many inappropriate practices existing in distribution systems of the cities in India. For example, in existing distribution system of many cities, it is observed that two or three existing reservoirs are observed to combinedly serve a single excessively large operation zone. This manual discusses how to correct such snags.
4) If DMAs within OZs are not properly established, water audit is not possible. Prioritisation of the leak repair programme is also not possible in absence of DMAs in the existing distribution system.

5) The technique of optimisation of diameters of pipes has been addressed in this document by introducing a new method. Uniqueness of this method is that it does not require any costly specialised software except the hydraulic model created by any easily available software.
6) One of the neglected areas in water supply is the equitable distribution of water in the distribution systems. Equitable distribution of water with designed pressure is the important aspect of 24×7 water supply. It is achieved by *Whole-to-Part* approach, in which two stages are involved: (a) equitable distribution from Master Balancing Reservoir (MBR) to service reservoirs and (b) equitable distribution from service reservoir to DMAs.
7) Equalisation of pressures (residual heads) at Full Supply Level (FSL) of service tanks is also a *grey* area. Equalisation of heads helps in effective and equitable supply of water to various service reservoirs in city by the transmission mains.
8) Currently, many cities are being transformed into *Smart Cities*. This manual describes the procedure to economically design pipelines on both sides of the roads by utilising these roads as boundaries for OZs and DMAs.
9) Pressure management strategies in Water Distribution Network is important. The methods of pressure management are discussed.
10) NRW computation is an important parameter in 24×7 systems. Estimating physical and commercial losses in the distribution system is an essential component of water balance in NRW reduction programme. This Manual discusses procedure to compute such losses. For this purpose, importance of connecting the meters and flow control valves to the Supervisory Control and Data Acquisition (SCADA) system is also discussed.

1.11 Composition of this Manual

The Manual intends to provide support so that all state governments and UTs to upgrade their water supply system to 24×7. The Manual is divided in three Parts: Part A, Part B, and Part C.

Part A: Engineering - Planning, Design and Implementation

Executive Summary
Chapter 1: Introduction
Chapter 2: Planning, Investigation, Design and Implementation
Chapter 3: Project Reports
Chapter 4: Planning and Development of Water Sources
Chapter 5: Pumping Stations and Pumping Machinery
Chapter 6: Transmission of Water
Chapter 7: Water Quality Testing and Laboratory Facilities
Chapter 8: Conventional Water Treatment
Chapter 9: Disinfection
Chapter 10: Specific Treatment Processes
Chapter 11: Pipes and Pipe Appurtenances
Chapter 12: Service Reservoir and Distribution System
Chapter 13: Water Meters
Chapter 14: Automation of Water Supply Systems
Chapter 15: Water Efficient Plumbing Fixtures
Chapter 16: Planning and Design of Regional Water Supply Systems

Part B: Operation & Maintenance

Executive Summary

Part C: Management

CHAPTER 2: PLANNING, INVESTIGATIONS, DESIGN AND IMPLEMENTATION

2.1 Introduction

Planning is defined as "defining objectives for a given period, designing various courses of action to achieve them and selecting the most practicable alternative from the various alternatives". In water supply systems, it is required to achieve the Service Level Benchmarks (SLBs) as set out by the Ministry of Housing and Urban Affairs (MoHUA), Government of India (GoI).

GoI launched Atal Mission for Rejuvenation and Urban Transformation (AMRUT) 2.0 in Oct, 2021 with a vision to make all cities' water secure and provide safe and adequate drinking water to all urban areas. Though GoI, State Governments, and Urban Local Bodies (ULBs) are making huge investments for providing safe and reliable water supply in urban areas, ULBs could not achieve the above said SLBs due to various reasons as discussed below. Water supplied at the household level is not meeting BIS (IS 10500:2012) and therefore, households adopt coping mechanism for improving water quality by using RO devices which are also not advisable as it is devoid of essential minerals.

As per the earlier Manual on Water Supply and Treatment published by the Ministry of Housing and Urban Affairs in 1999, all projects were planned, designed and implemented to achieve 24×7 pressurised water supply to supply safe and potable drinking water in adequate quantity, conveniently and as economically as possible. However, after implementation, the water supply systems were switched over to intermittent supply mode due to various reasons such as inadequate water resources, improper zoning, haphazard laying and tapping of pipes which are in unserved area and are not part of the design, low residual nodal pressure and lack of water meters etc.

Even though the earlier manual stated that the residual pressures should have been 7 m for a single storey building, 12 m for two storeys, 17 m for three storeys and 22 m for four storeys, most of the projects were designed with the residual pressure of 7 m or 12 m and operated in intermittent mode which results into contamination of water due to entry of dirty water into the pipeline during non-supply hours, high NRW and inequitable water supply.

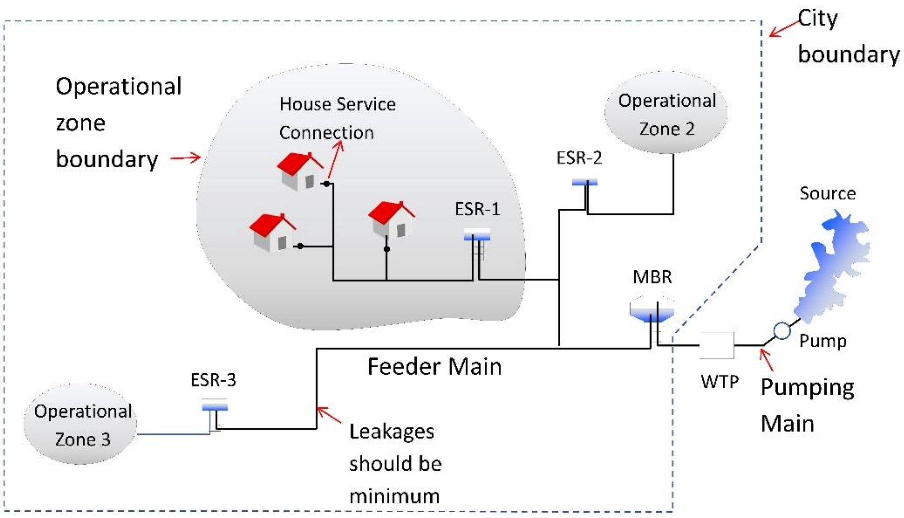
Drinking water quality is one of the biggest challenges in water sector of India. National Institution for Transforming India (NITI) Aayog in its Composite Water Management Index (2019) stated that eight million children (< age of 14) in urban India are at risk due to poor water supply. Infant mortality is the death of an infant before his or her first birthday. The infant mortality rate is the number of infant deaths for every 1,000 live births. The infant mortality rate for India (https://www.macrotrends.net/countries/IND/India/infant-mortality-rate) in 2022 was 27.695 deaths per 1000 live births.

Article 21 in 'The Constitution of India', 1949 states "Protection of life and personal liberty: No person shall be deprived of his life or personal liberty except according to procedure established by law". Thus, the right to access to drinking water is fundamental to life and there is a duty of the State, under Article 21, to provide clean drinking water to its citizens.

India is a party to the resolution of the UNO passed during the United Nations Water Conference in 1977: "All people, whatever their stage of development and their social and economic conditions, have the right to have access to drinking water in quantum and of a quality equal to their basic needs."

2.2 Essentials of 24×7 Pressurised Water Supply System

The city water supply scheme comprises of components such as collection at the source, a conveyance system in the form of a pumping main or gravity main for raw water and units for treatment, purification and transmission mains for treated water to the distribution system. A typical city water supply scheme is shown in Figure 2.1.



ESR: Elevated Service Reservoir; MBR: Master Balancing Reservoir; WTP: Water Treatment Plant

Figure 2.1: A typical city water supply scheme

Essentials of water supply scheme include adequate source which should be at least 95% reliable and dependable. 95% reliability and dependability mean that the source cater the needs of a city for at least 95% confidence intervals.

A proper water supply system consists of the following:

- The source of water should be free from contaminants
- Highly efficient transmission system for raw water
- Well maintained WTP
- Service reservoirs that do not get empty or overflowing
- Properly designed distribution system with well-established district metered areas (DMAs) to monitor and control NRW and ensure equitable water supply
- 100% metering with differential volumetric tariff

It is necessary to investigate, carry out survey, plan, design before execution of the scheme. Proper planning ensures that the scheme is implemented, commissioned operated and maintained within the scheduled time. The main steps involved in the implementation of the water supply project are shown in Figure 2.2.

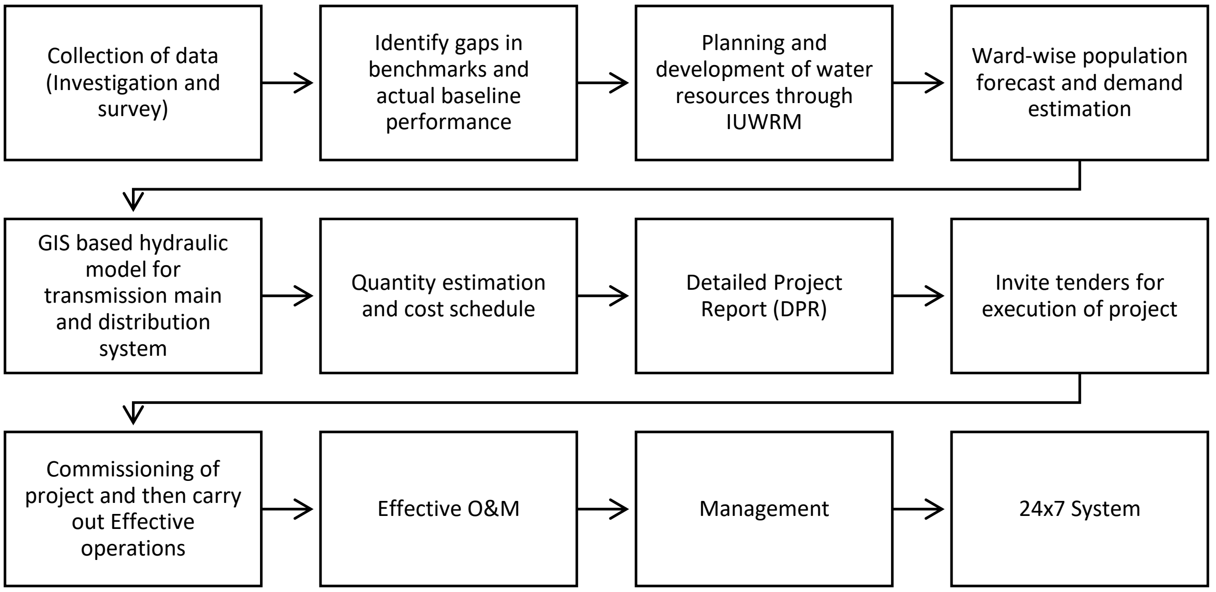


Figure 2.2: Main steps involved in the implementation of water supply project

Planning water supply involves the process of determining how water is proposed to be delivered to the consumers. Planning also requires assessment of any issues relating to water supply including protection of the sources. It also concerns the consideration of the water scarcity conditions and disaster management (emergency planning and response). Disasters can be natural (flood and drought), or human-made (chemical spill and sabotage). The city administration would respond to such conditions.

2.3 Vision, Goal and Objective

2.3.1 Vision

"All urban citizens and other user categories especially the poor and vulnerable should have access to adequate, safe and affordable "Drink From Tap" (DFT) facilities to meet personal hygiene and economic uses leading to sustained improvements in public health, well-being and economic productivity of urban areas through gradual conversion of intermittent water supply to a continuous 24×7 pressurised water supply and also covering uncovered areas in a phased manner in all cities and towns by 2047."

2.3.2 Goal

Gradual conversion and operationalisation of intermittent water supply to continuous 24×7 pressurised water supply and covering uncovered areas through a scientific and rigorous planning and implementation process to provide a safe and affordable water supply services to 100% urban citizens including poor and vulnerable by 2047.

2.3.3 Objective

Main objectives of city water supply system are: (a) to supply safe and potable water to the consumers as per drinking water quality as stipulated by BIS (IS 10500:2012); (b) to supply water in adequate quantity; and (c) to ensure equitable access with adequate pressure as equitable water supply brings affordability.

2.4 Proposed planning approach through DMA concept

DMAs are the building blocks of the 24×7 pressurised water supply scheme. Before the advent of DMA, identification of leaks in the distribution system was a difficult task. In early 1980s, DMA concept was first initiated in UK. With DMA, the problem of prioritisation of leaks was simplified. Since then, DMA methodology is being practised throughout the world. Bureau of Indian Standards code IS 17482: 2020 emphasises to adopt DMA concept to achieve 24×7 pressurised water supply system (PWSS with DFT). Thus, DMAs in distribution systems should be planned and designed for every city. The concept of DMA is prevalent in the developed countries and also in some of developing countries like some African and Southeast Asian countries.

In India, the concept of DMA has been propagated by CPHEEO, MoHUA by organising various international, national, regional and state level conferences/ workshops. The Ministry also published an Advisory on “Guidelines for Planning, Design and Implementation of 24x7 Water Supply Systems” in December 2021.

So far, the practice of DMA has been practised only in some states in India such as Karnataka, Odisha, Maharashtra, Tamil Nadu, Andhra Pradesh etc. Now, the awareness is being developed in many states and cities to adopt DMA concept. More than 600 cities and towns from about 27 States have reported that they are in process of formulation and implementation of projects based on DMA concept.

Cities such as Puri, Malkapur, Alnawar, Kundagola, and Thirthahalli have converted their intermittent system to 24×7 pressurised water systems for the entire city. Government of Odisha has also embarked DFT in 23 towns. Also, Nagpur, Coimbatore and Vishakhapatnam commissioned 24×7 PWSS with DFT partly. The other cities in Karnataka such as Hubli-Dharwad, Belgaum, and Kalburgi have partly commissioned their water supply to 24×7 pressurised system and have planned for full achievement.

The case studies of 24×7 water supply systems commissioned in case of Puri, Malkapur, Alnawar, Belagavi, Kalaburagi, Hubballi-Dharwad, Coimbatore, Pune, Nagpur, Visakhapatnam, Indi, Thirthahalli, and Shirpur cities is enclosed at **Annexure 2.1**.

The various ULBs mentioned above and few more ULBs who have implemented and are in the process of scaling up of 24×7 PWSS with DFT for pan city. They have achieved 24×7 supply by creation of DMAs, rehabilitation of existing water supply components, 100% replacement of the HSCs with a per capita cost in the range of Rs. 800 to Rs. 27,000 which largely depends on the condition of existing system, type of meters used and the cost of other water supply scheme components. The status of the 24×7 water supply projects and the details of the components can be referred to in the table at **Annexure 2.1**.

This manual strongly recommends planning and design of distribution system based on using the GIS and hydraulic modelling tools.

2.5 Reduction of NRW strategy

Non-revenue water (NRW) is defined as the difference of the quantity of water supplied and water billed. It comprises of the physical loss and the commercial loss. Physical losses are due to leakages in pipeline, inaccuracy of meters and overflow whereas commercial losses are due to theft, illegal connections, etc.

Many cities in India have NRW of more than 50%. Average NRW in Indian cities is 31%. This manual recommends NRW to be reduced to 15% for overall system and 10% for distribution system at DMA level. So, the strategy to reduce NRW is of paramount importance. The first step is to prepare GIS maps of the existing pipelines in the city and then prepare a hydraulic model. The boundaries of operational zones (OZs) and DMAs should be created on hydraulic model and the same maybe established by using isolation valves. The sub-DMA shall also be ascertained using isolation valves for monitoring NRW in case the DMA is not 100% metered.

When consumer metering is not done (which may be the case in most of the ULBs), the top-down approach of water audit should be adopted. Till 100% metering is achieved, the top-down water audit shall be carried out wherein quantum of water coming in the city can be known from the available pump registers and from the water billing data, the water consumption can be computed, the difference of water coming in and water consumed gives up approximate value of NRW.

The bottom-up water audit should be carried out when metering is done 100%. The bulk meter is installed at the entry point of the DMA. Every consumer should be metered and geo-tagged. The difference between the inflow of water coming in DMA and the quantity of water consumed in DMA gives the value of NRW of that DMA. Computation of NRW of all the DMAs should be carried out and the DMA with most leaking DMA should be tackled for leak identification and repair.

In case the metering is partially done, then water audit can be carried out in sub-DMAs. In this method, at least 10% of the customers in the sub-DMA should be metered. The flow in that sub-DMA can be measured by regular meter or by portable flow meter. This gives a sample value from which the NRW for the entire DMA can be extrapolated using statistical methods.

There are technologies that may identify the leakage areas when the values of flow and pressures (measured by pressure gauge at key locations) are fed to them. Other leakage methods such as noise co-relators can then be used to pinpoint the exact leakage spot. If the ULB desires to make quick leak identification of the pipelines, then some methods like helium gas, etc., can be used.

Replacing existing old leaking pipes and HSC shall result in NRW reduction substantial after formation of DMAs.

Advantage of NRW reduction programme is that once leaks are identified and repaired, water is saved and the saved water then leads to increased supply hours and in this way, NRW may be decreased to less than 15%.

2.6 Planning Objectives

The planning of water supply scheme aims at creating holistic/ comprehensive approach that help in effective water resource planning through Integrated Urban Water Resource Management (IUWRM) so as to achieve goal of converting existing water supply to 24×7 PWSS and covering uncovered areas to supply 24×7 pressurised water to every household meeting water quality standard as per provisions of IS 10500:2012.

The aforesaid objectives can be met by planning and designing of the water supply system by using DMA approach only to achieve 24×7 pressurised water supply. The planning based on DMAs concept has been standardised by Clause 8.5.2 of BIS 17482: 2020. Henceforth, ULBs shall plan and design urban water supply projects based on DMA approach which will enable them to improve the service delivery, control NRW from the present service levels and achieve 24×7 pressurised water supply. The Phase wise conversion of 24×7 PWSS is shown in Figure 2.3.

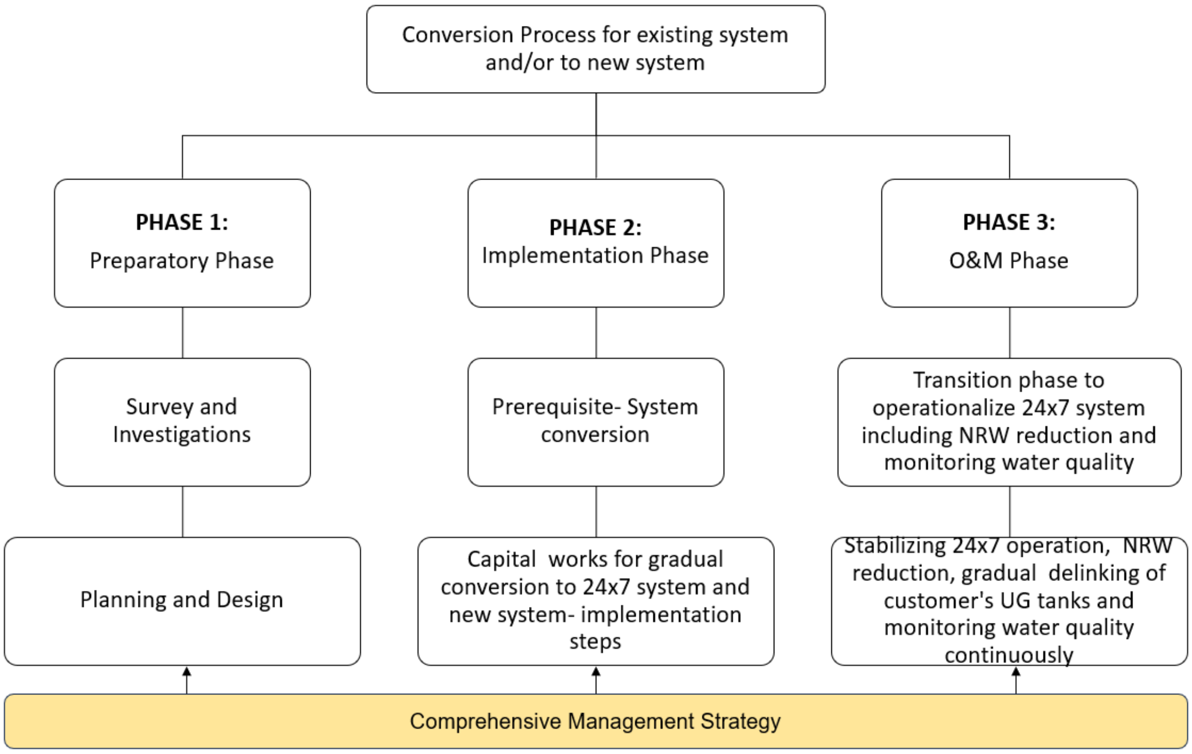


Figure 2.3: Conversion process of existing system to new system

2.7 Preparatory phase (Phase 1)

Preparatory phase includes of survey & investigation and planning & design of water supply schemes.

2.7.1 Preparatory Phase – Survey & Investigation

2.7.1.1 Survey for Elevations

A physical survey for elevation may not be required if the validated contours are generated using a 3D stereo-paired high-resolution satellite image. However, those cities who prefer to have a 2D satellite image shall carry out total station survey by taking levels along the city roads at 30 m chainage. GIS contours can be generated by the following methods.

a) Total station survey: Modern instrument consists of a theodolite with a built-in distance meter. Hence, it can measure angles and distances at the same time. It consists of a built-in emitter capable of emitting microwaves and infrared signals. Using the wavelength of these emitted waves, distance is calculated. Distance is calculated by multiplying the time taken to cover a certain distance by velocity.
b) GIS co-ordinates: Total station can measure the co-ordinates like X, Y, and Z or GIS northing, easting, and elevation of surveyed points.
c) In water supply projects, a surveyor conducts a survey along the city roads. With total station, generally, it records X, Y, and Z co-ordinates. Here, the city engineer should give directives to

record the northings and eastings along with the elevation of surveyed points along the road. These readings of northings, eastings and elevations in excel sheets are then used in GIS software to generate the shapefile of the points, which is then used to generate the GIS-based contours.

d) LIDAR: An elevation survey can also be conducted using Light Detection and Ranging (LIDAR) technology, which is a remote sensing method that uses light in the form of a pulsed laser.
e) Drones: Drones are also used to generate contours. Drones are used when the roads are not seen on the satellite images. Drones provide high-quality images. The drone flies along the flight path, and while passing, it takes precision images at two overlapping angles. Hundreds of high-resolution quality images are obtained and then processed by the appropriate software, which gives the Digital Elevation Model (DEM). DEM is then processed in GIS software to generate the contours. The contours thus formed should be validated by a Differential GPS (DGPS) survey.
f) DGPS-RTK: Differential Global Positioning System (DGPS) with real-time kinematics (RTK) can be used to make survey. All along the roads in city the ground elevations shall be recorded using DGPS. Using ground elevations GIS based contours are generated.
g) CORS: Recently, a Continuously Operating Reference Station (CORS) system is being used in the survey work of water supply of large cities. CORS is a network of RTK base stations that broadcast data usually over an Internet connection. A CORS comprises a GPS receiver operating continuously and antenna set up in a stable manner at a safe location (higher place like building top) with a reliable power supply for continuously streaming raw data. The centralised CORS station is usually connected to the multiple receivers (rovers) up to a distance of about 100 km. The levels recorded ensure uniformity which is suitable for large cities. The elevation and latitude and longitude co-ordinates are computed to an accuracy of 5-15 mm on the earth's surface.

2.7.1.2 Open Street Map

Open Street Map is a freeware tool using which we can get road edges, footprints of properties, railway tracks, water bodies, etc. However, Open Street Map is not used to generate contours.

2.7.1.3 Survey of Consumers

A consumer survey should be carried out to map the consumers in the distribution system. This survey should be planned for getting (a) requirement of consumer meters associated with various pipe diameter and type of use, e.g., residential, commercial, etc., (b) listing of suspected illegal connections and (c) connections from mainline which are to be shifted to lines designed for giving connections. Consumer survey provides information of consumer category, status of meters and current meter readings for billing purposes. GIS-based consumer geocoding provides information on the number of connections in each OZ of the service tanks, which determines the number of DMAs in the OZ. Information collected from this survey can be transferred to a GIS-based map. Geocoding with GIS co-ordinates of all the consumer meters is preferred.

The procedure for consumer survey is discussed in **Annexure 2.2**.

2.7.2 Investigations

Identifying Existing Pipelines and Condition Assessment

Identification of existing pipes is the necessary and most important activity both for augmentation and retrofitting in the existing system or a brand-new scheme.

For creation of hydraulic model, existing pipelines need to be identified and documented. Emphasis should be given to use existing pipe network in the model. It is extremely difficult to identify the existing pipeline as they are buried in ground and in most of the cities, database and maps of such pipelines are not available. There are five methods of detecting underground pipelines. These are (a) Manual digging pit, (b) Acoustic Detection Method, (c) Electromagnetic Induction Method, (d) Location of valves and (e) Ground Penetration Radar method.

All these methods of identifying existing pipelines are discussed in **Annexure 2.3**.

Various methods of condition assessment including that of robotics are as follows:

1) Robotic Pipeline Inspection
2) Inline Tethered Pipeline Inspection
3) External Non-Destructive Test (NDT) Techniques

All these methods of condition assessment are discussed in **Annexure 2.4**

2.8 Preparatory Phase - Planning & Design

2.8.1 Planning

The planning is required at various jurisdictional levels, i.e., for the urban areas of the country as a whole, the state level, regional level and community level. Though the responsibility of the various organisations in-charge of the planning of water supply systems can be different, they must function within the priorities mandated by the National and State Governments.

The water supply projects formulated by the various state authorities and local government agencies at present may not contain all the essential elements viz GIS maps, hydraulic modelling, equitable pressure, Supervisory Control and Data Acquisition (SCADA), etc. Also, different guidelines and norms are adopted by the States and ULBs; for example, population forecast, assumptions regarding per capita water supply, design period, size of zoning etc. Therefore, there is a need to specify appropriate norms for planning and designing to avoid the different approaches and maintain uniformity throughout the country.

The following aspects need to be considered in the planning and designing of water supply projects.

2.8.1.1 Achieving Benchmarks

The targeted SLBs for water supply notified by MoHUA in 2008 are shown in the Table 2.1

Table 2.1: Targeted service level benchmarks for water supply services

S. No.	Performance indicator	Targeted Benchmark
1	Coverage of water supply connections	100%
2	Per capita supply of water	135 LPCD
3	Extent of metering of water connections	100%
4	Extent of NRW	15%
5	Continuity of water supply	24 hours
6	Quality of water supplied	100%

7	Efficiency in redressal of customer complaints	80%
8	Cost recovery in water supply services	100%
9	Efficiency in collection of water supply-related charges	90%

While planning the water supply scheme, the ULB shall carry out the survey of its city/town and find out the baseline parameters of the performance indicators. The gaps between the benchmarks and the baseline parameters shall be worked out and the detailed project report (DPR) shall be prepared to bridge the gaps so that the benchmarks, as shown in Table 2.1, shall be attained. Some of the SLBs such as 24×7 water supply, 100% metering, control of NRW and quality of water supply shall be considered as Key Performance Indicators (KPIs) in the tender document. In addition to the above, residual nodal pressure shall be included in the tender document as KPI.

All water supply projects should be implemented with the objective to achieve the aforesaid SLBs and monitor the same throughout the design period.

2.8.1.2 Planning Considerations

Planning long-term requirement for sustainable water supply in India is a big challenge due to the complexity of the system and rapid growth in population and water demand. The challenge further increases as the city water sources are becoming distant due to the non-availability of nearby reliable and adequate water sources, thus increasing the project's cost. Engineering decisions are required to specify the area and population to be served, the design period, per capita rate of water supply, other water needs in the area, the nature and location of facilities to be provided, the utilisation of centralised or decentralised treatment facilities and points of water supply intake and wastewater disposal. Projects have to be identified and prepared in adequate detail in order to enable timely and proper implementation.

A detailed long-term planning is needed to decide the number of phases and phase-wise expansion of the water works synchronising with the expansion of the urban area. Working capital cost required, interest charges, period of loan repayment and water tax should be given due consideration.

2.8.1.3 Planning and Development of Water Sources

Integrated Water Resource Management (IWRM) is defined as a technique that encourages co-ordinated land and water development and management in order to maximise economic and social welfare in an equitable manner and is needed for comprehensive planning of river sub basin and groundwater sources. In a river sub basin, there are number of cities dwelling on the bank of the same river. State Water Resource Departments/Irrigation Departments need to compute the water balance for entire river sub basin including groundwater sources in consultation with State Groundwater Board/Department which will give an available balance of water for planning of water resources for various consumptive/non-consumptive uses. ULB need to carry out the study of IUWRM for a city which is a subset of IWRM. IUWRM needs the water availability, input variable and various demands in a city as an output variable based on the water demand for population and other non-domestic needs and availability of water from surface and groundwater sources, recycled water, rainwater harvesting, sea water, etc. Thus, using IUWRM, ULBs need to prepare city water balance for sustainable planning of the city water supply to ensure 95% dependability and reliability of water sources for a design period of 30 years as per requirement for water supply project. The outcome of IUWRM tells us whether the city has enough water or is in deficit for catering its water needs. If the water balance is in deficit, the city has to comprehensively plan for addressing the deficit/ gap in water by recycling of water, rainwater harvesting, etc. Details of IWRM and IUWRM including City Water Balance Plan is discussed in section 4.13 & 4.14 of Part A Manual.

City engineers to ensure that the city has a perennial sustainable water source with 95% dependability. This includes evaporation losses for the projected population of the ultimate stage with designed per capita supply. Water resources department make planning of dams and the city can take this information from them.

Dedicated express feeder with standby arrangement for electric substations at pumping stations at headworks and at Water Treatment Plant is mandatory. The work of electric lines shall be done from the corresponding electricity board. Electricity board shall ensure that they shall not give electric connections to other consumers from this dedicated express feeder. The cost of the express feeder should be included in the project cost.

2.8.1.4 Water Security

Urban water security does not merely mean developing water sources and supply water at every household in urban areas, but it is globally defined as the dynamic capacity of the water system and water stakeholders to safeguard sustainable and equitable access to adequate quantities and acceptable quality of water that is continuously, physically, and legally available at an affordable cost for sustaining livelihoods, human well-being, and socio-economic development, for ensuring protection against waterborne pollution and water-related disasters, and for preserving ecosystems in a climate of peace and political stability.

2.8.1.5 Water Quality and Quantity

The objective of Water Works Management is to ensure that the water supplied is free from pathogenic organisms, clear, palatable and free from undesirable taste and odour, of reasonable temperature, neither corrosive nor scale forming and free from minerals that could produce undesirable physiological effects. The establishment of minimum quality standards for public water supply is of fundamental importance in achieving this objective. The water to be handled may vary both in quantity and quality and the degree of treatment required changes seasonally, monthly, daily and sometimes, even hourly. The public health engineer may use his ingenuity to mitigate the variations in quantity by the provision of storage, which may be drawn upon during peak demand. Variations in quality can be managed by provision for the introduction of suitable process adjustments in the WTP.

It is the responsibility of the ULB/Water Boards/PHED to supply water with adequate quantity and required pressure and acceptable quality meeting drinking water supply standards at every household as per the Tables 1 to 5 of the BIS code IS 10500:2012 which are shown in **Annexure 2.5**.

2.8.1.6 Strategy for Improvement of Drinking Water Quality

Following strategy shall be adopted for improvement of drinking water quality:

i. **Contamination of drinking water in pipeline during non-supply hours:** It occurs during non-supply hours in intermittent water supply system when the pipeline is empty which attracts outside contaminants, thus contaminating water. The strategy is to provide continuous water supply with adequate pressure so that the entry of outside contaminants is prevented.
ii. **Contamination of drinking water in customer underground tanks:** Customers construct underground (UG) storage tanks due to interrupted supply prevalent in the distribution

system. In the past, the UG tanks were constructed with brick masonry where the joints in bricks are porous. Thus, not only the water leaks but it also allows outside contaminants to enter. Therefore, consumer UG tanks should be discouraged for the buildings up to the three storeys. The strategy is to provide continuous potable water supply with residual pressures of 17-21 m in case of Class I and II cities and 12-15 m for other and then subsequently remove UG tanks gradually. However, for high rise buildings, waterproof UG RCC/HDPE tanks are recommended with annual cleaning using chlorine.

iii. **Contamination of water in pipelines through ferrule points of HSCs**: As per various studies conducted, about 70-80% leakages occur at ferrule points and they become the point of contamination. This manual emphasises use of standard quality ferrules and pipes in addition to employing the services of licensed skilled plumbers for giving HSCs.

iv. **Contamination of raw water sources due to discharge of untreated wastewater**: Progressively over the past, the wastewater (partially treated or untreated) has been discharged into the water bodies. As the wastewater effluent and the pesticides leached from the agricultural fields discharged into the water bodies contain various contaminants including micro-pollutants, Endocrine Disrupting Chemicals etc., it is important to adopt high degree of treatment to protect the public health. Therefore, the conventional water treatment processes are ineffective to treat such variety of contaminants. The degree of treatment for raw water sources may be required to be enhanced with a judicious combination of appropriate treatment technologies by making additional investments. The suggested line of treatment options for contaminated surface water sources have been provided in Chapter 8.

2.8.1.7 Water Conservation

Rising demand for water in urban communities due to population increase, commercial and industrial development and improvement in living standards is putting enormous stress on the water resources. Not only the quantity of extractable freshwater resources is being depleted but also the quality is deteriorating. The problem is further aggravated due to the over-abstraction of ground waters and/or indiscriminate use of surface water bodies for the discharge of municipal and industrial untreated wastewaters. It has, therefore, become essential to initiate measures for an effective and integrated approach to water conservation.

2.8.1.8 Increasing the Water Availability, Supply & Demand Management

The measures required to increase the water availability involve augmentation of water resources by storing rainwater on the surface or below the surface. Surface storage is usually contemplated either in natural ponds, reservoirs, and lakes or artificially created depressions, ponds, impounding reservoirs, or tanks. Subsurface storage of water is affected by constructing subsurface dykes, artificial recharge wells, etc. For storing subsurface water in rocky areas, several techniques have been developed indigenously like Jack Well Technique, Bore Blast Techniques, Fracture Seal Cementation. These techniques have been deployed to improve porosity, storage volume as well as interconnectivity between fractures/fissures and other types of pores. Artificial recharge of ground water may be considered in areas that are suitable for such purpose.

Water supply management aims to improve the supply by minimising losses and wastage and reducing NRW in the transmission mains and distribution system. A robust performance monitoring system should be planned to secure the quantity and quality of water including reduction of NRW by adopting the methodology of water balance suggested by International Water Association (IWA.) The NRW can constitute a significant fraction of total water supplied in poorly constructed and managed water transmission and distribution systems. Measures like detection, control and prevention of leakage, metering of water supply, installation of properly designed water efficient taps and prompt action to repair and maintain distribution system components should be adopted.

Water demand management involves measures that aim at reducing water demand by optimal utilisation of water supplies for all essential and desirable needs. It can also be done by enforcing differential tariff based on the volumetric consumption. It focuses on the identification of all practices and uses of water more than the functional requirement. The appropriate use of plumbing fixtures, such as low volume and dual flushing tanks in place of conventional cisterns that conserve water should be encouraged. Practices like the recycling and reuse of treated wastewater should be promoted as mandated under AMRUT 2.0 to conserve fresh water sources. In many cities, apartments are mandated to treat wastewater and reuse in their premises.

2.8.1.9 Planning of OZs and DMAs

The city should be divided into pressure zones based on the GIS based contours of the city within the jurisdiction of each WTP. The city should be further divided into Operational Zones (OZs) within a pressure zone based on the contours with each OZ defining the minimum and maximum pressure. There shall be at least one OZ for each service tank. After determining the optimum boundaries of OZs of all existing service tanks, new service tanks should be planned in the unserved areas. Care shall be taken to see that the maximum ultimate population of each OZ shall not exceed about 50,000 or 10,000 connections in plain areas and for hilly areas, maximum population per OZ should be about 30,000 or 6,000 connections. Each OZ shall be divided into sub zones which are called as DMAs. Each OZ shall have not more than four DMAs. Each DMA shall have the number of connections in the range of 500 to 3000 in plain areas and 300 to 1500 in hilly areas and all DMAs shall be hydraulically discrete (isolated) for which zero pressure test (Refer Section 12.12.2) shall be planned. Each DMA shall be connected to its respective service reservoir by a common branch pipe connected to the outlet of service reservoir. On the branch pipe connecting to each DMA an arrangement comprising of isolation valve, bulk meter and flow control valve (FCV) should be made. The bulk meter and the FCV shall be connected to the SCADA through the Remote Terminal Unit (RTU).

In some cases, land for construction of service tanks may not be available and very few service tanks but larger capacity has to be planned, in such cases the number of District Metering Areas (DMAs) may be more than 4 as per the terrain conditions. This may be also applicable in the area where population is saturated.

In saturated/high density population areas, where land is a constraint, construction of service reservoir for catering OZ with 50,000 population, the norm of 50,000 population per OZ shall be relaxed and ultimate population up to 75,000 to 100,000 shall be considered in OZ with proper justification. However, maximum no. of household connections shall be restricted to 3000 by increasing the suitable no. of DMAs.

The design of various components of OZs under DMAs are provided under preparatory phase design mentioned in Clause 2.7.2.2.

2.8.1.10 Location of Water Supply System Components

Though the distribution layout and the sources of supply and their development methods are important in placing the different units like headworks, transmission mains, WTP, overhead or underground storage tank, pumping stations, pressure reducing valves, flow control valves, etc. for optimal and economical utilisation, factors like topography, soil conditions and physical hazards should also be taken into consideration. Hillside construction may have an advantage in accommodating the head loss in the plant without excessive excavation. Wet sites must be dewatered and structures may have to be designed considering the hydrostatic uplift. On the soils having low bearing capacities, structures may need to be placed on piles or rafts. Rocky sites may require costly excavation.

Flooding is a common hazard for the treatment plants and pumping stations located near rivers and other surface water bodies. The highest flood level observed at the site should be taken into account and the treatment plant and pumping station structures shall be built at least two feet above the high-water mark. Irrigation and Flood Control Department should be contacted for the flood warning system.

2.8.1.11 Automation

Mechanisation, instrumentation and automation are becoming more and more common in water works and distribution network and this should also be considered in planning the system, subject to local availability and maintenance facilities.

Automation replaces and serves the functions that cannot be performed efficiently by manual operations, such as the removal of the sludge from sedimentation tanks etc. Instrumentation involves the installation of various kinds of devices and gauges for monitoring and recording plant flows and performance. Automation combines instrumentation and mechanisation are required to monitor water quality parameters, levels, pressures and flow etc., in headworks, WTP, service reservoirs and distribution network.

2.8.1.12 Service Building

Considerable attention is to be given to the service building required at treatment works and pumping stations such as houses, offices and laboratories, storerooms, chemical house, pump house, etc. In moderate climates, only operating units need to be protected against rain and sun, while in adverse climates, complete protection of all the units is advisable.

2.8.1.13 Other Utilities

Provision needs to be made for facilities such as electricity, water supply, drainage, roadways, parking areas, walkways, fencing, telephone facilities and other welfare services such as housing for operation and maintenance personnel.

2.8.1.14 All Season Roads

Headworks, WTP, sumps, Balancing Reservoir and Elevated Service Reservoirs (ESRs) should be accessible in all seasons by road. All pipelines of principal transmission main feeding MBR and sump should be laid along all-season road and transmission mains from MBR should be preferably laid along all season road or at least cart tracks which are accessible even during monsoon.

2.8.1.15 Planning of Big Zones (group of several OZs)

Large cities have generally more than one WTPs. Each such WTP has its own jurisdiction or supply area and each of them contains several service reservoirs and thus a number of OZs. The following considerations shall be given to holistically plan such big zones.

(i) Demarcate each every WTP on the GIS map of the city.
(ii) Create base map of jurisdiction of each WTP. The base map comprises of road edges, footprint of each property, water bodies, land use polygons of residential, commercial, industrial areas, etc.
(iii) Carryout elevation survey along the roads and create GIS contours in the area under consideration.

(iv) Create pressure zones using contours/elevation points. Pressure zones visualise high altitude area and low-lying areas of the city in different colour codes. Pressure zones help in designing the OZs and its feeder mains.
(v) Carryout consumer survey of domestic and commercial customers.
(vi) Show existing pipelines after identifying them also show existing service reservoirs.
(vii) Create network of the existing pipelines using GIS based hydraulic model.
(viii) Assign ground elevations and demands to all the nodes of the existing pipelines.
(ix) Determine optimum boundary of each of the existing service reservoirs and mark unserved areas.
(x) There should be one location both for existing and proposed new service reservoir (in phases depending on design) for one OZ.
(xi) Plan new service reservoirs in the unserved areas.
(xii) Plan new pipelines in unserved areas to make 100% coverage.
(xiii) Assign demand to the nodes of new pipelines.
(xiv) Design transmission mains from clear water sump of WTP to each service reservoirs (both existing and new)
(xv) The Manual recommends 30yrs. Design period for service reservoirs. If in case two service reservoirs are planned (one for 15 years and another for next 15 years) due to land constraints then the transmission main shall also be connected to such tanks.
(xvi) Design distribution system network using hydraulic model.

In this way, big command areas of WTP shall be planned. Detailed flow chart for planning OZs/DMAs of the command areas is provided in Figure 2.6.

2.8.1.16 Planning of Existing Large Size Service Reservoir

Sometimes, the large-sized service tanks are constructed in difficult terrain where the land for construction is not available. In such situations, the number of DMAs may be more than four. However, size of DMA shall by maximum 3000 connections. A separate pipe shall be branched from the common outlet of the service tank leading to each DMA. Necessary isolation valve, bulk meter and FCV shall be installed at the entry point of each DMA.

If some of the DMAs are located at lower ground elevations, necessary pressure reducing valve (PRV) shall be installed to regulate the nodal pressure in such DMAs.

If the large-sized service tank is located at high altitude, then nodal pressures would be more. Suitable pipes in the distribution shall be planned to sustain higher nodal pressures.

However, if the larger sized service tank is located at flat terrain (which should be discouraged) and if the residual nodal pressures are less, then VFD pump may be planned to increase nodal pressures.

2.8.1.17 Planning of Ground Water Schemes

In many urban areas which depend on ground water sources, water from tube wells is directly pumped into distribution system. This practice of pumping water from number of tube wells directly into distribution system shall be discouraged as it has following demerits:

- There will be interruption of water supply during power failure or any breakdown.
- Direct chlorination in the pipeline will provide less contact time and the households near tube well will get pungent smell due to high concentration of chlorine which may also affect their health.

- There will be wear and tear of pumps due to back pressure when many pumps are directly connected to the distribution system. In certain cases, the flow may be from multiple directions as water is pumped from multiple tube wells.
- There will be heavy leakage from pipes due to high pumping head.

Therefore, it is recommended to pump the water from tube well into common clear water reservoir (CWR) and then to the service reservoir. Capacity of clear water sump may be considered as 25% of capacity of ESR planned. From the service reservoir, water is supplied to the distribution network. Total NRW in ground water sources is 11%, out of which 10% may be allowed in the distribution system.

It must be ensured that the water quality of every tube well should meet the physical and chemical parameters stipulated in BIS IS 10500:2012. If not, appropriate treatment for removal of hot spot parameters such as salinity, iron, fluoride, arsenic, etc., shall be given and then taken to CWR.

2.8.1.18 Data Required in Planning Phase

(i) General data

General data required are as follows:

a) census population data for the last three to five decades;
b) daily per capita supply in litres at the consumer end (LPCD);
c) supply hours for the design of pipelines up to ESRs, i.e., for rising/transmission mains;
d) capacity and staging height for ESR and side water depth (SWD) (difference between maximum water elevation and minimum water elevation in the tank);
e) residual nodal head;
f) demand management by consumer meters;
g) water tariff - a tool for demand management;
h) losses in the system;
i) valves and meters;
j) land required for planning.

(ii) **Collection of Available Data for both Existing and New Schemes**

The implementing Agency/ULB should collect the necessary information/ data which is required to prepare the DPR. DPR should contain background of the project, population projection, water demand, DMA formation, design of various water supply components, standards and specifications, bill of quantity, etc. Agency/ULB is required to collect all relevant data and prepare the DPR, if required, the same may be outsourced. The following information is required:

1) Details of all sources and their 95% reliability and dependability
2) Ward boundaries with ward-wise population of the latest census year, the population of the census year
3) Base maps: GIS based shape files of road edges, streams, property footprints, GIS based contours, etc.
4) Details of existing distribution system and other water supply components including WTP etc.
5) Existing valves and its location (If valves are corroded and defunct, they should be either removed or replaced)

6) Pumping station details, including principal mechanical and electrical plant infrastructure specifications, i.e., details of pumps, motors, starters, transformers, etc. of their actual duty details, age and status
7) Details of reservoirs such as ESR, Ground Service Reservoir (GSR), and Master Balancing Reservoir (MBR), including capacity and validated operating levels, including staging height, present life and repair details
8) Details of bulk supply of water
9) Status of the statutory clearances
10) Permission of land availability
11) Arrangement of financial resources

2.8.1.19 Land Required for Water Supply Infrastructure

Even though the water treatment units are designed and initially made functional for an intermediate stage of 15 years, land should be kept available for the ultimate stage (30 years after base year) and future expansion.

The land for elevated service reservoirs shall be earmarked for 30 yrs. In case sufficient land is not available for service reservoir, then direct pumping to distribution system using VFD pumps may be considered to reduce the footprint area where adequate standby power backup.

City planners should earmark the land required for water supply infrastructure and its expansion in the ultimate stage in the master plan of the city for a minimum of the next 30 years. As cities are planning for DMAs/OZs, necessary land may be earmarked as per the requirement by the ULBs.

The city planner should consult and ascertain land requirement for water infrastructure and incorporate the same in the City Development Plan (CDP)/ Master Plan and ULB should amend the bylaws accordingly.

When the land for water supply infrastructure is not available, the city planners should allow development of water infrastructure over or below recreational amenities or parks, stadiums, etc. Such planning is shown in **Annexure 2.6**. The authorities may have to amend the planning rules/ by laws to implement such arrangement.

2.8.1.20 Base Maps

Creating base maps using GIS includes the following:

(i) Satellite Image

A satellite image of the city with 0.5 m resolution should be obtained. The satellite image has two formats - 2D satellite image and 3D stereo paired image. Those cities whose terrain is relatively flat can go for procuring 3D stereo paired images so that they generate seamless contours of 1 m intervals for the entire city.

It is observed that most cities carry out surveys by different agencies with different benchmarks. Thus, when one tries to integrate the contours, they are not seamless. This difficulty can be overcome by procuring a 3D stereo paired satellite image. The city administration can obtain this image from National Remote Sensing Agency (NRSA). After obtaining the image, the contours can be generated with appropriate photogrammetric software. The contours generated shall be validated by carrying out DGPS survey. DGPS is attached with several satellites and gives accurate level of the spot. Normally, one reading per square kilometre is taken to validate the contours.

If a satellite image is not procured, then the designer can use the online service of the GIS software, which makes online satellite images available.

(ii) Digitisation of Features

Digitisation is the process of converting information into a digital format. When the image is scanned the scanner converts it to an image file, such as a JPG or bitmap. On digitisation, information is obtained, which makes it easier to preserve, access and share. Digitisation is required for the base maps as it is used as background drawing in network software. Digitisation of properties in a city is used to map the consumers in GIS.

There are some of freeware like Open Street Map which provide digitised shape files of road centreline, footprints of house properties, water bodies, etc.

(iii) Landmarks

Landmarks can be created from the satellite image, Google Earth etc.

(iv) Existing Water Infrastructure

Transmission and distribution pipelines, tanks, etc., are created by several ways, as mentioned in the Advisory on “GIS Mapping of Water Supply and Sewerage Infrastructure” published by MoHUA in 2020.

(v) Mapping of existing pipe network

Existing pipe network including isolation valves, air valves, flowmeters, stand posts, etc., should be mapped on the GIS base maps of the city.

a. The existing pipeline can be known from the “as-built” completion drawing of existing scheme. As-built drawings are the completion drawings of existing scheme when the scheme is commissioned. During handing over of the scheme to ULB, these as built drawings are also handed over.
b. The existing pipelines can be made known by interacting with the group of residents, mechanic, plumbers, retired operators, valve operators, meter readers, etc., along with them the utility engineer can interact with the local people to enquire about the location of alignment of pipeline and approximate year of laying.
c. Pipe locators can be used to assess the pipe alignment wherever required. This work has been successfully carried out in Coimbatore city.
d. In some of the cities, the existing pipeline are identified by ground penetrating radar (GPR). Wherever possible, this method can be used.
e. Wherever possible, in cities, the adequate number of trial pits can be taken to identify the attributes of existing pipe network.
f. Existing pipe can be considered in the design or in hydraulic model only if their location on map, material, diameter, year of laying is known. Otherwise, the hydraulic model should be created using the data of existing pipes whichever is available at least to complete the model. This is a continuous process and cannot be done 100% at the initial stage. To begin with, the hydraulic model should be prepared using above and it should be continuously updated after knowing the additional data.

The methods of identifying existing pipelines are discussed above in Section 2.7.2.

2.8.1.21 Contour

Contours should be generated by conducting a survey.

2.8.1.22 Planning Tool

A Geographic Information System (GIS) is the most effective tool used in planning water supply schemes. GIS is defined as "a system designed to capture, store, manipulate, analyse, manage, and present or display spatial or geographically referenced information, i.e., data identified according to their locations". GIS information required is elaborated in the Advisory on, "GIS Mapping of Water Supply and Sewerage Infrastructure," which is available on Govt. of India's web site https://mohua.gov.in/ pdf. GIS can put information on maps. Here, information means things in the real world that are organised into layers. For example, to comprehensively depict its distribution system, a city requires various information like street data, building data, pipes data, and contours data which are organised in the layers. Integrated data is displayed as a combined map.

Box-1: What is Shape file?

A shape file is a simple, non-topological (shared boundary is stored once for each polygon) format for storing the geometric location and attribute information of geographic features. Geographic features in a shape file can be represented by the primitive geometric shape of points, lines, or polygons (areas).

Why Shape files?

Shape file stores non-topological data and attribute information for spatial features. Feature's geometry is stored as a shape having vector co-ordinates like latitude and longitude.

Since, processing of the topological data structures is avoided, the shape files are supposed to be efficient for rendering and requires less memory space and easy to read and write.

2.8.1.23 Creation of Land Use Map of City

Land use maps of a city comprise of the spatial information/data of the various physical land uses like the residential area, areas of commercial activity, transportation, parks and gardens, forest land, etc. These land use coverages are generally provided in City Development Plans (CDP). Normally, Town and Country Planning Department creates such CDP maps, which are in GIS format. ULB must get such maps in consultation with Town and Country Planning Department. The map of CDP, if available in hard copy, should be collected and georeferenced. After the process of geo-referencing, the polylines of roads, buildings, etc., are exported to form the shape file of the different types of the land use.

2.8.1.24 Population Density using GIS Maps

Following steps may be followed:

(i) **Determining Population Density of Wards:** A GIS ward map of all the wards of a city is prepared. The polygons of different wards are digitised and a shape file of the boundary of all the wards of the city is created.

(ii) **Ward-Wise Land Use Area**: Though the map showing all types of land uses for the entire town is available, it is necessary to find out different types of land use areas for each individual ward. To divide the different type of land use areas for each ward, the 'split' command from the GIS software can be used. Two overlapping shape files - (i) land use map and (ii) wards are used to form overlapping layers. After executing the split command, shape files of each ward with corresponding land use areas are obtained. Information from these shape files after the split command is collected.

(iii) **Projected ward-wise population by Equivalent Area Method:** Objective is not only the total population of the city, but its ward-wise distribution and computation is required for allotment of the present and future water demands to the nodes of the distribution network. In the large pipe network of the distribution system of water supply, future demand needs to be assigned to hundreds of the nodes. Manual exercise of this demand allocation to nodes is prone to error. In most of the softwares, the demand is given using the population density map which is based on the land use maps. Therefore, land use maps are required prior to the creation of population density maps.

Since the population density of each ward with respect to land use is to be found out, it is required to find out the equivalent area of each ward. While determining equivalent area, the general factors - such as 100% for residential, 25% for public and 10% for industries and agriculture must be used.

An illustrative example of the projected ward-wise population by the equivalent area method is incorporated in **Annexure 2.7**.

2.8.2 Design

The comprehensive planning and design norms are discussed in the following paragraphs and summarised in Table 2.7. Sustainable O&M practices of continuous (24×7) PWSS are summarised in Table 2.8.

2.8.2.1 Design Period

The design period of the water supply scheme depends on the life of the components sharing a significant proportion of the cost as well as the difficulty in augmenting them. The projects must be designed normally to meet the requirements over a 30-year period (Handbook on Water Supply and Drainage (SP 35: 1987) of Bureau of Indian Standards) after their completion and commissioning. The time lag between design and completion of the project should also be considered, which should not exceed two years for small and medium size projects and five years for large size projects. The 30-year period may, however, be modified regarding certain components of the project depending on their useful life, the facility for carrying out extensions when required and the rate of interest so that excessive expenditure in due course of time is avoided. Necessary land for future expansion should be acquired in the beginning of the project. Where large tunnels and aqueducts are involved entailing significant capital outlay for expansion, they may be designed for ultimate project requirements. Where there is a possibility of failure such as the collapse of steel pipes under vacuum which may put the pipeline out of commission for a long time or the pipe location presents hazards such as floods, ice, mining, etc., duplicate lines may be necessary.

Redundancy should be factored into the design plan and included in cost-benefit analysis to evaluate trade-off of system failures.

Stages in design period: Stages involved are defined as follows:

- Base year: means the proposed date of completion of the scheme.
- Intermediate stage: is computed as base year + 15 years.
- Ultimate stage: is computed as base year + 30 years.

However, different components of the water supply system are designed to work satisfactorily for different periods, as shown in Table 2.2.

This manual suggests consideration of using existing infrastructure which is in good condition while designing the proposed scheme. Rehabilitation could extend some of the items listed in Table 2.2. They should be considered in the design of the system. For example, when WTP is to be planned for 15 years, civil structures of the existing WTP, after assessing their useful condition, must be considered. So, usefulness of existing structures would not be jeopardised.

Table 2.2: Design period in years

S. No.	Items	Design period in years
1	Storage by impounding reservoirs/dams/barrage/weir	50
2	Headworks (intake, jack well or canal intake)	
	(a) Pump house (civil works)	50[!]
	(b) Electric motors and pumps	15
3	Groundwater source (tube wells/bore well/dug wells)	
	Tube wells, bore well	15
	Life of pumps and for ground water	15
	Life of pumping main for ground water	30
4	Water treatment units	15*
5	Channels and pipe connection to several treatment units in WTP	15**
6	Raw water, clear water conveying mains and Pipes in Distribution system	30[!!]
7	Clearwater reservoirs at the WTP, balancing tanks	15*
8	Service reservoirs (overhead or ground level)	30[#]
9	Civil work of pump house for direct pumping	30
10	Pumping machinery for direct pumping	15

! The spaces in the pump pit and pump house need to be designed for all working + standby pumps for both stages.

* Land allocation to be made for 30 years.

** The pipe sizes shall be computed considering the 20% overloading in the WTP, i.e., over and above the intermediate demand. However, since, Aeration fountain, Inlet channel including parshall flume, flash mixer and flow distribution box to clarifiers are common for the present and future stages above components though constructed in present stage need to be designed for flow of ultimate stage.

!! WTP after 15 years should be located in the same premises. However, if it is located at different place which is away from the existing, then the pipeline shall be designed for the capacity of the respective WTP.

The ESR is recommended to be designed for 30 years because of the following reasons:

- It should be ensured that each OZ should be served by one ESR.
- In most of the projects, it is observed that initially one ESR is designed and constructed for initial 15 years as per previous guidelines. Another ESR was to be designed for next 15 years. However, in almost all the projects, additional ESR is not constructed and only one initial ESR

with 15 years demand capacity is serving the OZ of 30 years demand. This has vitiated the hydraulics and the nodal pressures dropped thus forcing the system to be resorted to intermittent water supply scheme.

- Even two ESRs are proposed for intermediate and ultimate stages, the pipe network has been designed initially for the ultimate demand is now to be reorganised after 15 years when the second ESR is to be constructed. Changing network after 15 years is virtually difficult task and not practised at all in the field.
- The capacity of ESR will be one-third of the ultimate stage demand and will ensure 24×7 continuous water supply throughout the design period of 30 years.

2.8.2.2 Population Projections

The first step in the water supply scheme planning process is to quantify current and future population projection and then the corresponding water demand.

General considerations: The design population will have to be estimated with due regard to all the factors governing the future growth and development of the project area in the industrial, commercial, educational, social and administrative spheres.

Any underestimated value will make the water supply system inadequate for the purpose intended; similarly, the overestimated value will make it costly. Special factors causing sudden emigration or influx of population should also be foreseen to the extent possible. Change in the population of the city over the years occurs and the system should be designed considering the population at the end of the design period. Factors affecting changes in population are:

- increase due to births
- decrease due to deaths
- increase/decrease due to migration
- increase due to annexation

The present and past population records for the city can be obtained from the census population records. After collecting these population figures, the population at the end of the design period is predicted using various methods suitable for that city considering the growth pattern followed by the city.

- Demographic Method
- Arithmetical Increase Method
- Incremental Increase Method
- Geometrical Increase Method
- Decreasing Rate of Growth Method
- Graphical Method
- Logistic method
- Method of Density
- Curvilinear method

Various methods of population forecast are discussed in **Annexure 2.8**.

However, the ULB/ parastatals should finalise total population for immediate and ultimate stage in consultation with Town and Country Planning Department before preparation of DPR of the water supply project. Total population thus arrived shall be judiciously distributed ward wise by ascertaining trend of growth, i.e., ward wise population density for immediate and ultimate stage for designing the distribution network as detailed in section 2.8.1.24 of Part A Manual.

2.8.2.3 Per Capita Supply

Piped water supplies for communities should provide adequately for the following as applicable:

a) domestic needs such as drinking, cooking, bathing, washing, flushing of toilets, gardening and individual air conditioning
b) institutional needs
c) industrial and commercial uses, including central air conditioning
d) firefighting
e) requirement for livestock
f) minimum permissible NRW

2.8.2.4 Factors Affecting Consumption

The following factors affect water consumption:

a) Size of City: Water demand increases with an increase in the size of the town or city. The water demand increases in terms of water use, road cleaning, maintaining parks, etc.
b) Characteristics of Population and Standard of Living: The water demand depends directly upon the habits and economic status of the consumer. A big city with higher living facilities will have higher water demand than a town with lower living facilities. Slum areas of large cities have low per capita consumption. A person staying in an independent bungalow consumes more water compared to a person staying in a flat. The person's habit also affects consumption; the type of bath, i.e., tub bath or otherwise and material used for washing, etc., also affect per capita consumption.
c) Industries and Commerce: Industrial and commercial activities increase water demand in the area. The type and number of different industries also affect consumption. The water consumption in the industry or commerce varies considerably depending on the processes included and the size of the industry.
d) Climatic Conditions: With a rising temperature and uneven rainfall, the water demand will also get affected. In hot weather, the consumption of water is more compared to that during cold weather. The issue of climate change is to be considered while developing a water demand forecast model to achieve sustainable water supply management.
e) Metering: The consumption of water is less when supply is measured by the water meters compared to that when the water charges are on a flat rate basis.
f) Variation in water demand: The hourly variation takes place on a day when the water demand is at its peak while it drops down in other hours of the day. Mornings and evenings are associated higher residential use because of getting ready in the mornings and returning home in the evenings.

2.8.2.5 Recommendations

In the Code of Basic Requirements of Water Supply, Drainage and Sanitation (IS: 1172-1993, Reaffirmed 2007), a minimum of 135 LPCD has been recommended for all residences provided with a flushing system for excreta disposal. The breakup of water requirements is shown in Table 2.3.

Table 2.3: Average water use per person per day in urban area

S. No.	Purpose	Quantity (LPCD)
1	Drinking	5

S. No.	Purpose	Quantity (LPCD)
2	Cooking	5
3	Bathing	50
4	Toilet flushing	30
5	Washing utensils	15
6	Washing the house	10
7	Washing of clothes	20
	Total	135

It is well recognised that the minimum water requirements for domestic and other essential beneficial uses should be met through the public water supply systems which are defined in the following paras. Other needs for water, including industries, etc., may have to be supplemented from other systems depending upon the constraints imposed by the availability of capital finances and the proximity of water sources having adequate quantities of acceptable quality which can be economically utilised for municipal water supplies.

Based on the objectives of full coverage of urban communities with easy access to potable drinking water to meet the domestic and other essential non-domestic needs, the following recommendations are made:

(i) Recommended per capita Water Supply Levels

The earlier manual (1999) suggested to adopt 150 LPCD for all metro and mega cities, 135 LPCD for cities/towns that have sewerage system or are contemplating to have such system and 70 LPCD for the towns that do not have sewerage system. This manual recommends LPCD values as shown in Table 2.4. The Class I & II cities and towns should plan for water supply projects considering a per capita water supply of 150 and 135 LPCD as proposed below (Table 2.4) and should take up underground sewerage systems within three years of commissioning of water supply schemes.

The other towns which are planning for water supply projects considering 135 LPCD should also take up underground sewerage system within three years from the commissioning of water supply projects. In case towns which have source constraints and are not contemplating sewerage system within the next 5 years, they can restrict per capita water supply to 100 LPCD for water supply projects and plan for decentralised sewerage facilities/ on-site system with reuse facilities as recommended in Sewerage Manual.

Table 2.4: Recommended per capita water supply levels for designing schemes

S. No.	Classification of towns/cities	Recommended Maximum Water Supply Levels (LPCD)
1	Cities/ towns with a population of less than 10 lakhs (0.1 million)	135
2	Metro and Mega cities having a population of 10 lakh (1 million) or more	150

Note:

- Supply should be at the consumer end. This means 15% system losses shall be added to the demand.

- The domestic demand does not include bulk requirements of water for semi-commercial, commercial, institutional and industrial purposes. Demands due to commercial (malls, hotels etc), institutional and industrial purposes must be assessed separately through consumer survey and duly extrapolated for different stages.
- Such demands should be assigned to the nearest pipe/nodes of the pipe network in the distribution system.
- Semi-commercial demands include micro industries, market, shops, vegetable market, traders, hawkers, non-residential tourists, picnic spots, religious places, etc.
- In the absence of consumer survey, the present demand due to semi-commercial to the tune of about 5-10% of intermediate demand (domestic) may be considered depending on the nature of the town. The semi-commercial demand for intermediate and ultimate stages may be calculated considering an increase of 1% per year on the initial semi-commercial demand.
- Fire demand should be added to domestic demand proportionately.

(ii) Requirement of Floating Population

The rate of supply for the floating population (CPHEEO, 1999) should be as follows (Table 2.5):

Table 2.5: Rate of supply for floating population

S. No.	Facility	Litres per capita per day (LPCD)
1	Bathing facilities provided	45
2	Bathing facilities not provided	25
3	Floating population using only public facilities (such as market traders, hawkers, non-residential tourists, picnickers, religious tourists, etc.)	15

The data on floating population/ tourists shall be obtained from the tourism department of the State Government.

In the absence of such data, floating population may be considered as percentage of ultimate stage (30 years) population as below:

- Class I cities: 2-5%
- District HQ: 2-3%
- Hill Stations: 5-10%
- Seaside cities:5-10%
- Small towns: 1-3%

However, ULB can increase/ decrease floating population with proper justification on case-to-case basis.

(iii) Institutional Needs

The water requirements for institutions should be provided in addition to the provisions indicated in Table 2.6, where required, if they are of considerable magnitude and not covered in the provisions already made. The individual requirements (CPHEEO, 1999) would be as shown in Table 2.6.

Table 2.6: Requirement of water for institutions

Sl. No.	Institutions	Litres per head per day
1	Hospital (including laundry)	
	(a) No. of beds exceeding 100	450 (per bed)
	(b) No. of beds not exceeding 100	340 (per bed)
2	Hotels	180 (per bed)
3	Hostels	135
4	Nurses' homes and medical quarters	135
5	Boarding schools / colleges	135
6	Restaurants	70 (per seat)
7	Airports and seaports	70
8	Junction Stations and intermediate stations where mail or express stoppage (both railways and bus stations) is presided	70
9	Terminal stations	45
10	Intermediate stations (excluding mail and express stops)	45 (could be reduced to 25 where bathing facilities are not provided)
11	Day schools / colleges	45
12	Offices	45
13	Factories	45 (could be reduced to 30 where no bathrooms are provided)
14	Cinema, concert halls, and theatre	15

(iv) Fire Fighting Demand

Prior to computation of fire requirements of OZ, it is necessary to compute the fire requirements for the entire city using following formula:

Fire requirement for entire city = $100\sqrt{P}$ (m^3/day)

Where P is the population of the intermediate stage (15 years) of the entire city in thousands.

$$\text{Fire Requirement of OZ} = \left(\frac{\text{Intermediate population of OZ}}{\text{Intermediate population of the entire city}}\right)(\text{Fire requirement of the entire city}) \quad \text{... Eq 2.1}$$

In case the service reservoir is designed for ultimate stage the word "intermediate" shall be replaced by "Ultimate".

It is desirable that one-third of the firefighting requirements of each OZ form part of the service storage. For this purpose, the outlet of the tank supplying water for normal operation should be kept just above this storage so that the capacity provided for mitigating fire is always available. There should be fire outlet at the bottom of the tank that can be opened when an instance of fire occurs as well as at the time of cleaning the tank.

The balance requirements may be met out from secondary sources. The high-rise buildings should be provided with adequate fire storage from the protected water supply distribution. Also, there is a remote possibility that the fire occurs at multiple places, hence nearby ESRs can also be used for firefighting requirement.

The location of fire hydrants should be decided in consultation with Fire Department. However, arrangements for filling vehicles of fire brigade should be provided at each ESR. The pressure required for firefighting would have to be boosted by the fire engines.

(v) Total demand

In addition to domestic demand, fire demand, commercial demands (hotels, lodges, hospitals, markets, etc.) and institutional demand (schools, colleges, offices, theatres, etc.) duly extrapolated for different stages (base year, immediate and ultimate) should be added as point loads to the respective nodes in the distribution system.

Total demand should be computed by adding the following losses:

Total losses in the system (surface water) should not exceed 15%. The indicative break-up of losses is shown in Figure 2.4.

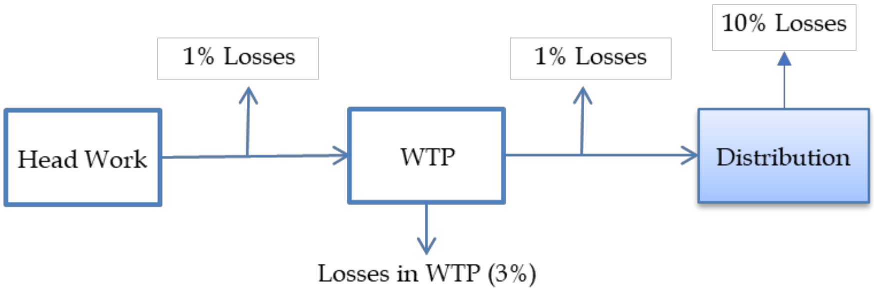


Figure 2.4: Indicative break-up of losses

a) Headworks to the inlet of WTP should not be more than 1%.
b) In WTP, losses should not be more than 3%.
c) Outlet of WTP to Various ESRs losses should not be more than 1%.
d) Sometimes, the location of WTP is close to headworks and sometimes it is close to the city boundary. Hence, (a) and (c) above put together shall not be more than 2%. However, if (a) and (c) together is more than 20 km then total loss should be considered at the rate of 1% per 10 km, instead of 2%.
e) In a distribution system, losses should not be more than 10%. (With 24×7 Water Supply project with 100% metering, NRW is expected to be reduced. Hence losses should not be > 10%).

For ground water where water is directly supplied to distribution system and WTP is not part of the system, the total loss should not exceed 11%.

2.8.2.6 Pressure requirement

Pressure requirements are discussed in Table 2.7 of design norms. Piped water supplies should be designed on continuous 24 hours basis to distribute water to consumers at adequate pressure at all points. Intermittent supplies are neither desirable from the public health point of view nor economical.

2.8.2.7 Formation of OZ and DMAs Based on Pressure Zones

A pressure zone is defined (www.usbr.gov/gp) as "the area bounded by both a lower and upper elevation, all of which receives water from a given hydraulic grade line (HGL) or pressure from a set water surface."

Objective of providing pressure zones is to provide water to customers in adequate quantity in an efficient manner. By forming pressure zones high and low elevation zones are separated, hence cost of pumping and O&M cost can be lowered. Pressure zones are formed using GIS techniques as follows:

a) Add shape file of city boundary on the online satellite image. Online image is available on GIS software.
b) Add shape file of GIS contours.
c) Using GIS tool, form the land polygons called as “Topo-to-Raster”
d) Alternatively, if the survey is carried out along the roads by taking levels at fixed chainages, say 30m, then these points can be mapped on the online GIS data layer. Using GIS tool Inverse Distance Weighted (IDW), surface/polygons shall be formed and different elevation polygons shall be demarcated with colour code in GIS.
e) Elevation range is marked.

The resulting image is shown in Figure 2.5. Pressure zones are shown in different colours.

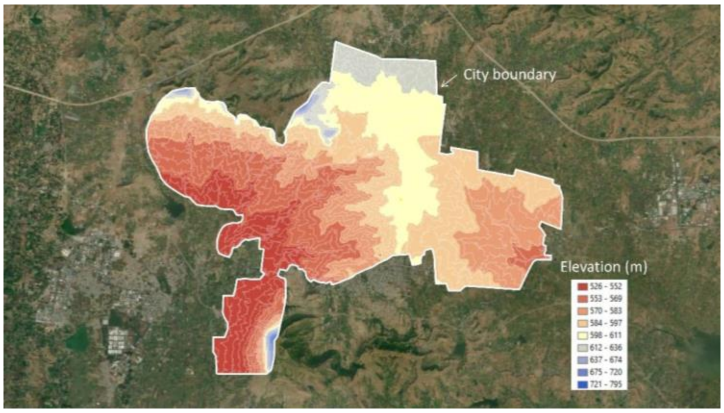


Figure 2.5: Pressure zones

2.9 Logical Flow Diagram for Switching Over Process

Switching over process from intermittent supply of existing system to 24×7 water supply requires reengineering and refurbishing water system considering aspects of DMA for their optimal utilisation. The process can be referred by the concerned levels.

(a) Broad Summary (for administrators and senior level engineers)
Broad Summary of planning and implementation stages are shown in Figure 2.6. This table should be referred by the administrators and senior level engineers.

(b) Detailed Steps (for consultants and junior level engineers)
Detailed Steps of planning and implementation stages are shown in Figure 2.7 (a) to (h). This table should be referred by the consultants and junior level engineers.

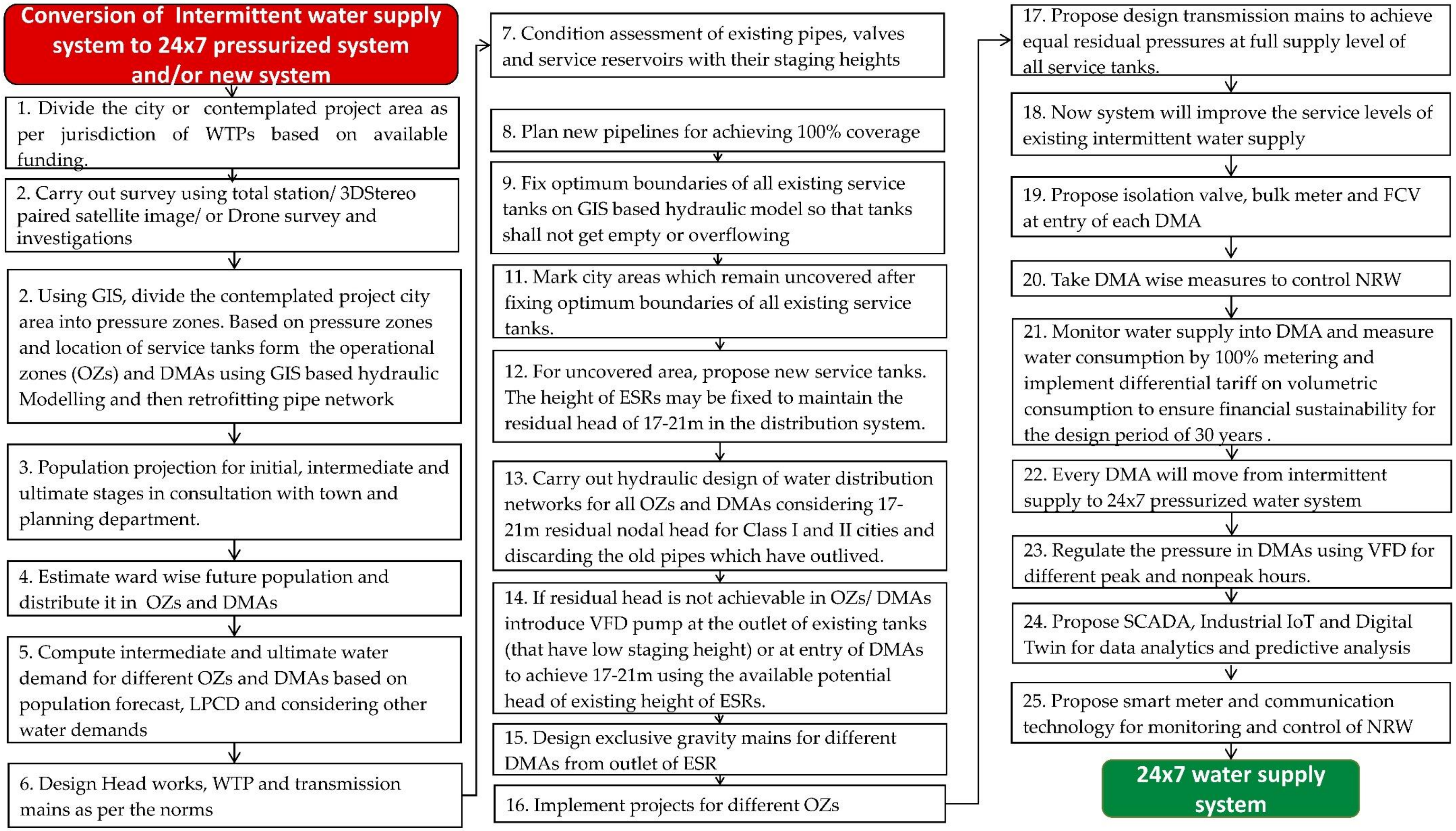


Figure 2.6: Summary of conversion of intermittent water supply to 24×7 pressurised water supply and/or new system for administrators and senior level engineers, they should refer Tables 2.7 and 2.8.

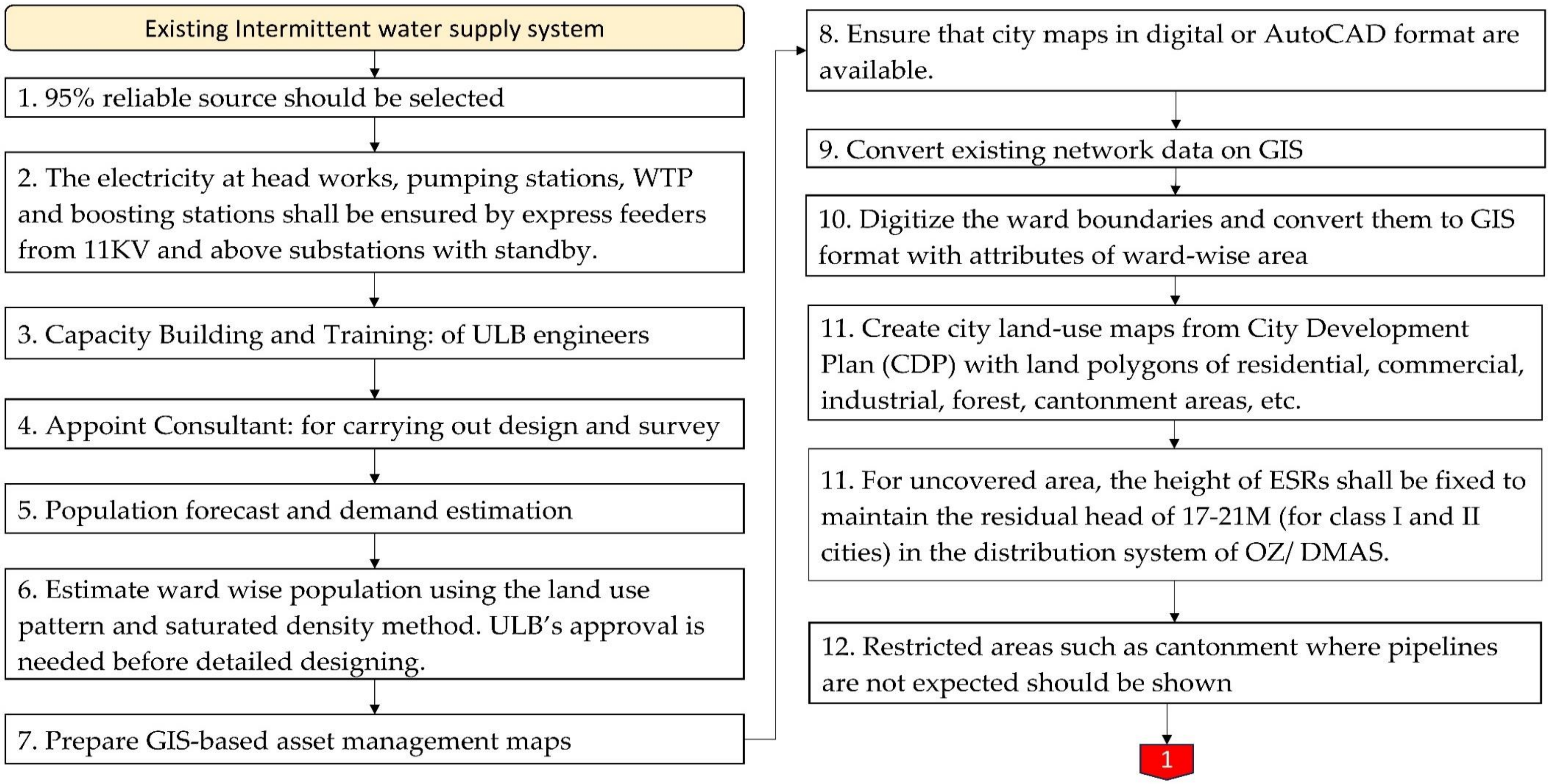


Figure 2.7 (a): Planning and design of conversion of intermittent water supply to 24×7 pressurised systems for consultants and junior level engineers, they should refer to Tables 2.7 and 2.8.

Note: Figure 2.7 consists of Figures 2.7 (a) to (h) which are connected by the connectors shown in red pentagons.

1

13. Geo-reference all AutoCAD maps and converted them to GIS format

14. Carry out condition assessment of the existing pipelines and valves. Only the usable existing pipes with their attribute database should be considered in the design

15. Carry out ground elevation survey along roads of city using suitable survey method

16. From road elevations, "topo-to-raster" surface shall be created and then GIS based contours shall be generated. Thus, pressure zones (in different colours) of the entire city are formed, which mark high and low elevations areas of the city

17. Existing service tanks feeding high-level areas should be marked which should be fed by separate transmission mains so that only the needed quantity of water is pumped to a high level and energy cost is optimized

18. Based on the survey of the existing pipelines, the GIS based hydraulic model should be prepared showing the existing pipelines and existing service tanks. Ground elevation and demand should be given to all nodes

19. Considering maximum serving capacity of the existing service tanks the optimum boundaries of all the existing service tanks should be determined. Maximum serving capacity is computed three times that of the volume of the existing service tank. Optimum boundary of the service reservoir ensures that the tank do not get empty or overflow

20. Once optimum boundaries of all existing service tanks are marked in hydraulic model, some pipes and nodes may be seen which are unserved part of the distribution system.

21. New Service Tanks: These tanks shall be planned and designed in the unserved areas

2

Figure 2.7 (b): Continued

2

22. Proposed new pipelines shall be designed in the OZs of existing and new service tanks. Existing and proposed service reservoirs (in unserved areas) should also be considered in the hydraulic model

23. Only two GIS-based hydraulic models are prepared for the entire city- (i) for the entire distribution system and (ii) for raw/treated transmission mains. If the city is exceptionally large and is divided into excessively big zones as per jurisdiction of each WTP, then the two models can be prepared each jurisdiction of WTP (one for transmission mains and another for distribution system).

24. In Distribution System:

a. The operational zones, one per service tank should be planned. DMAs should be planned as sub zones of each operational zones.

b. The installation of the DMAs should be included as an item in the DPR.

c. Adequate provision of items under retrofitting of existing pipelines (such as cutting of the pipes, providing dead ends, replacing pipes etc.) should be made.

d. 100% consumer metering should be ensured with differential tariff based on the volumetric consumption.

e. Leakage occurs at joints of municipal pipe with house service connection (HSC) if ferrule connection is not made. Hence, at such places, ferrule connections must be made by the skilled plumber of the municipality. For this purpose, an item of replacing HSC should be incorporated in the DPR.

25. Operational Zone: Only one operational zone (OZ) should be planned for each service reservoir. One operational zone may have maximum of 4 district metered areas (DMAs), except in the terrain where service reservoirs are difficult to construct due to non-availability of land. OZ shall be designed to cater the ultimate stage population up to 50,000 or 10,000 connections in plain areas. For hilly areas OZ may be designed for ultimate stage population up to 20,000 or 4,000 connections. Number of connections in one DMA should preferably be in the range of 500 to 3000 in plain areas and 500-1500 in hilly areas.

26. All the DMAs are hydraulically discrete (isolated)

27. For forming the boundaries of OZ and DMAs, natural boundaries like nalla/ roads/river/ railway line be used. Proper retrofitting of existing pipe network is required.

28. The distribution pipe network is for existing as well as newly proposed pipes. 100% coverage shall be ensured.

3

Figure 2.7 (c): Continued

3

29. If the existing pipes are too old and joints leak, then these pipes need systematic replacement

30. The minimum diameter of the distribution network should be 100 mm. For hilly areas, the minimum diameter of distribution system is 80 mm; however, for a housing societies pipes of suitable diameter 63-80 mm be considered from the municipalities pipe.

31. Roads having a width of 6 m or more, pipes are to be laid on either side of the road. This can also be done economically while deciding the boundary of DMA

32. From outlet of the service tank, exclusive pipelines shall be laid up to entry point of each DMA (Figure 2.8). No consumer connection shall be given from such pipelines.

33. Isolation valve, bulk meter and flow control valve (FCV) should be planned on the entry pipe of the DMA. In hilly area where terrain is uneven, PRVs can be used to regulate the pressures in place of FCVs. Integrated control valve can also be used.

34. All such valves, especially the flow control valve at entry of each DMA should be connected to the SCADA system (for cities with population more than 10 Lakhs) and it should be ensured that each DMA must get water as per design

35. A VFD pump having the capability of controlling both the flow and pressure shall be designed and installed on the outlet of the service tank of the OZ having low residual pressures (short staging height of ESR).

36. A VFD pump can take advantage of positive suction head (due to available staging height of ESR provided that the tank is not empty) and provides flexibility to generate 17 to 21m in Class I & Class II cities and 12 to 15m for other cities.

4

Figure 2.7(d): Continued

4

37. After running the hydraulic model for the OZ, the nodal pressures are checked for required heads (17-21m for Class I & Class II cities and 12-15m for other cities). The velocity of flow in pipes shall not be more than 2.5m/s. In hilly regions, as well as the pipeline joining ESR with main pipeline the velocity may be as high as 4.3 m/s. The reasons for adopting the pressures in the range of 17-21m are as follows:

a. In most of the developed countries during maximum hourly demand, minimum nodal pressures have been adopted in the range of 21m to 35m and the maximum pressures are in the range of 56 to 70m. Of course, they provided water by direct pumping.
b. In earlier manual 17m nodal pressure has been recommended for building with 3 storeys and 22m up to 4th floor.
c. In order to get rid of consumer's leaking and contaminated underground storage tanks, water should reach up to 3rd floor.
d. When 21m nodal head is achieved, there may be some leakages in the old network. This may be due to old pipes and improper joints. During this transition period there will be drop in pressures owing to the leakages and the pressure may come down to about 17m.
e. The drop in pressures may be observed as reduced water level (standing water depth) in service tanks during peak hours.
f. Considering above Sr. (d) and (e), about 4m drop in pressures occurs and effective nodal pressures may come down from 21m to 17m. To account for this, design nodal pressure should be 21m.

38. The maximum allowable pressure at any ferrule points other than the critical nodes in an operational zone in the existing distribution network shall be the field test pressure of the existing pipes.

39. All items with item rates required for making OZ and DMAs hydraulically discrete should be incorporated in the cost estimate of DPR. The items should include retrofitting existing pipelines {such as cutting the pipes, providing dead ends (should be discouraged as far as possible), replacing pipes, etc.}. This is necessary for the installation of the DMAs

40. After retrofitting, the road needs reinstatement for which a separate item should be included in the DPR

41. House service connections for individual consumers should not be given on the pipes in distribution network having more than 200 mm internal diameter. For this purpose, additional rider pipes may be shown (but not compulsory) and included in the design of the distribution system

5

Figure 2.7 (e): Continued

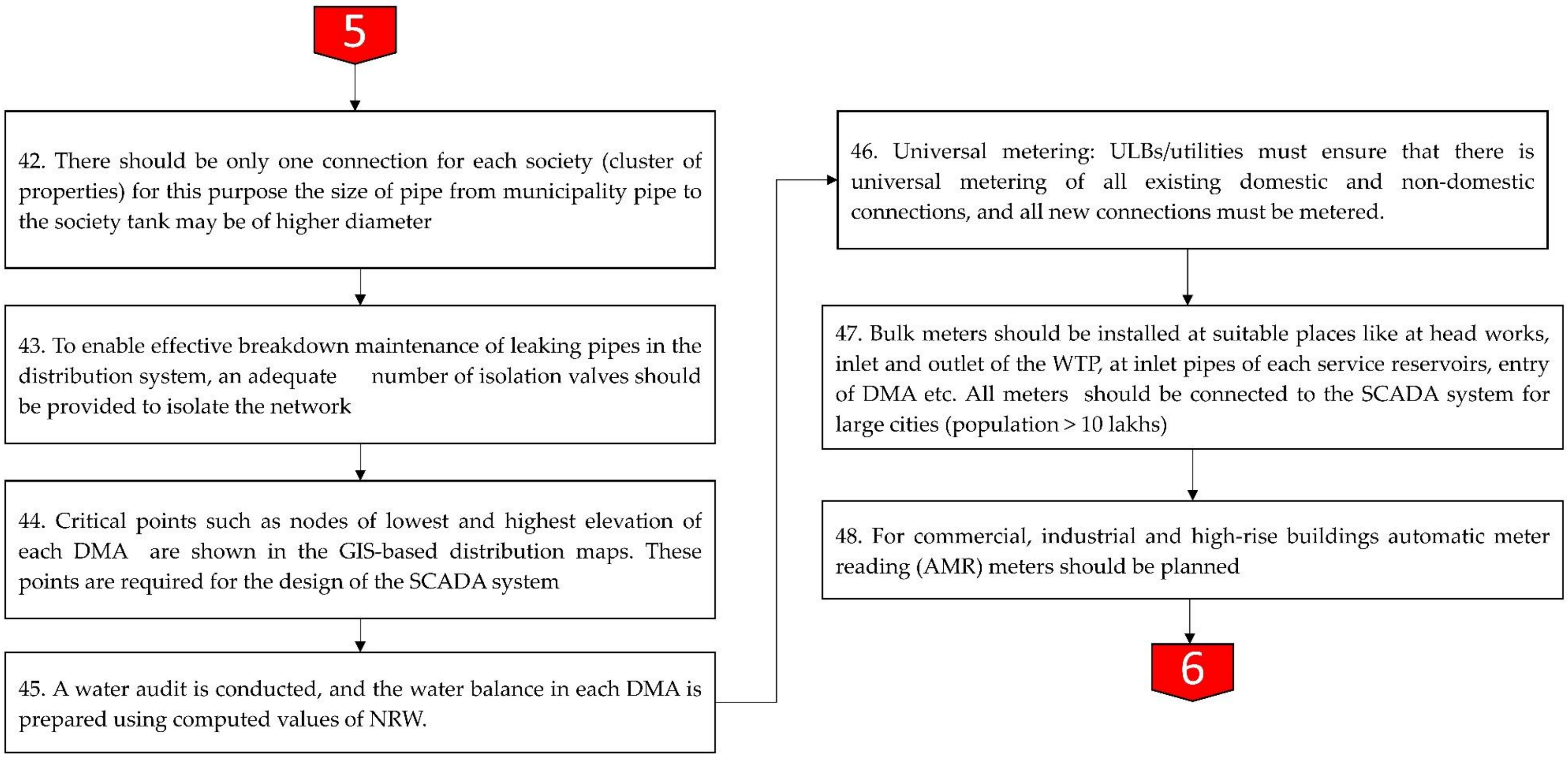


Figure 2.7 (f): Continued

6

49. Connections: City engineer to observe that- in most of the developed countries during maximum hourly demand, minimum nodal pressures have been adopted in the range of 21m to 35m and the maximum pressures are in the range of 56 to 70m. Of course, they provided water by direct pumping.

(a) City bylaws should be amended to meet the requirements as follows.

(b) As every family is to be supplied water through house connection, no new stand post should be given.

(c) Existing stand posts should be removed and converted to connections to households with the meter by formulating an operational zone-wise time-bound program by the ULB.

(d) If the distribution pipes are non-metallic and valves are rusted, a systematic plan of replacing them is prepared. Adequate budget provision is made for this purpose. Replacement of these existing old non-metallic pipes should be planned by running the GIS-based hydraulic model.

(e) Leakage occurs, in most of the cases, at the joint of municipality pipe with house service connection (HSC) made of substandard material and the connection is made by the non-skilled plumber. Hence, at such places, ferrule connections shall be made using quality products and materials by the skilled plumber of the municipality. For this purpose, an item for replacing HSC should be incorporated in the DPR.

(f) Clause 9.5 of the IS 2065: 1983 reaffirmed 2022 states that the ownership of consumer meters shall rest with ULB.

(g) All consumers shall be metered, and the estimated cost of meters should be provided in the DPR.

50. Valves: City engineer to observe that-

a) Required control valves (FCVs/ PRVs for hilly areas) are planned for the designed distribution of water.

b) All such valves, especially the flow control valve at the entry of each DMA, are connected to the SCADA system (wherever possible), and it is to be ensured that each DMA gets water as per design.

51. Disaster management: system should be designed for each city. For this purpose, the GIS based hydraulic model should be prepared and interlinking of the various water works within the city should be planned

52. Miscellaneous: City engineer to observe that-

a) WTP, MBRs, ESRs, and pumping stations are located on all-season roads, and the necessary roads are provided in DPR along with the cost so that pipelines up to the city boundary are laid on all-season roads.

b) Wherever possible, Break Pressure Tank (BPT) should be planned. The inlet and outlet of BPT/ MBR should be placed at the same level.

c) Energy audit of pumps should be carried out.

d) Cities having population of more than 1 lakh should have their own meter repair and calibration cell and adequate spare part should be stocked.

7

Figure 2.7 (g): Continued

7

53. Transmission Mains: Main pipelines being the lifeline of the city, all pumping mains shall be internally lined metallic pipes. Gravity pipelines feeding the service reservoirs and are to be laid within municipal boundary shall also be preferably metallic/ proper suitable pipes

54. The transmission mains should be designed properly by optimizing the capital and energy cost. The transmission mains are so designed that the residual head at Full Supply Level (FSL) of each reservoir is nearly equal (preferably 3m). This arrangement should be ascertained by connecting all the service tanks with the SCADA system. The details are given in Chapter 6 in Part A of this manual

55. The rising mains and transmission mains should have capacity to supply water for next 30 years and should not leak anywhere

56. *Water Hammer*: Water hammer devices should be installed at proper locations. For this purpose, the pumping mains must be analyzed for surge. In such analysis, the hydraulic grade line representing minim pressure shall be preferably above the pipe top which ensures that there is no negative pressure. If it is below pipe invert and if negative pressure is less than -0.9 Kgf/cm^2, then pipe needs surge protection

57. In cities where water is pumped directly into distribution system, the distribution system should be checked for surge analysis

58. NRW cell should be created in each ULB

59. NRW Management: It is required to reduce NRW to the extent of 15%.

60. ULB to undertake NRW assessments and adopt the NRW reduction strategy and action plan incorporating following:

- Active leakage control (at distribution mains, storage tanks, and service connection points),
- Identify illegal connections
- Regularizing illegal connections,
- Minimizing free supply.

61. After carrying out all above steps of planning and design (in A), actual implementation exercise of transforming to 24x7 system should be carried out.

24x7 Pressurized water supply

Figure 2.7 (h) - End of Figure 2.7

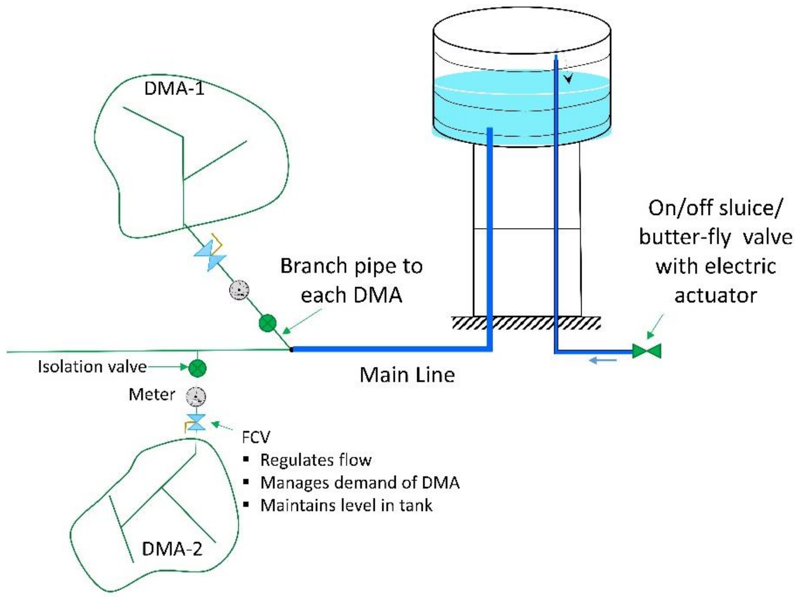


Figure 2.8: Separate branch pipe to each DMA

2.10 Implementation phase (Phase 2)

2.10.1 Prerequisite

2.10.1.1 System Conversion

(i) Removing Public Stand Posts

Stand posts are provided for supply of water to low-income group who cannot afford independent connection. However, a lot of water wastages have been observed at stand posts as supply through stand posts are free and no-one is accountable for wastage at such locations. Therefore, the stand posts are required to be converted/ eliminated to and individual connections to be with metered supply by providing subsidy.

(ii) Replacement of Faulty Consumer Meters, Faulty Service Connections

Metering is essential to levy consumers based on the quantity of water utilised. If the existing mode of charging is based on flat rate, then it should be changed, and consumers should be charged based on quantity of water utilised. Therefore, new meters should be installed.

A survey should be carried out to check the status of each meter, connection through ferrule and status of service line up to meter. Service lines are normally of galvanised iron (GI). GI pipe gets rusted fast when it is buried underground. Studies have shown a lot of water loss at service connection and in-service lines. Leaky lines should be repaired or replaced depending upon the status of pipeline. Consumers should also be advised to check the pipeline beyond meter and get leak repaired if any.

(iii) Regularisation of Illegal Connections

Illegal connections are one of the major causes for high NRW. Their identification is difficult and once identified the present process of regularisation is a big task because it involves penalties for illegal use for the period for which water has been used illegally. A proper strategy is needed for regularisation of illegal connections. First, it is necessary to identify the suspected connections. During consumer survey, the survey team may follow the steps shown in Figure 2.9 to roughly identify the suspected illegal connections. If any family is identified as suspected of an illegal connection, meter reader should regularly make physical verification of that suspected consumer and try to bring him into the billing cycle.

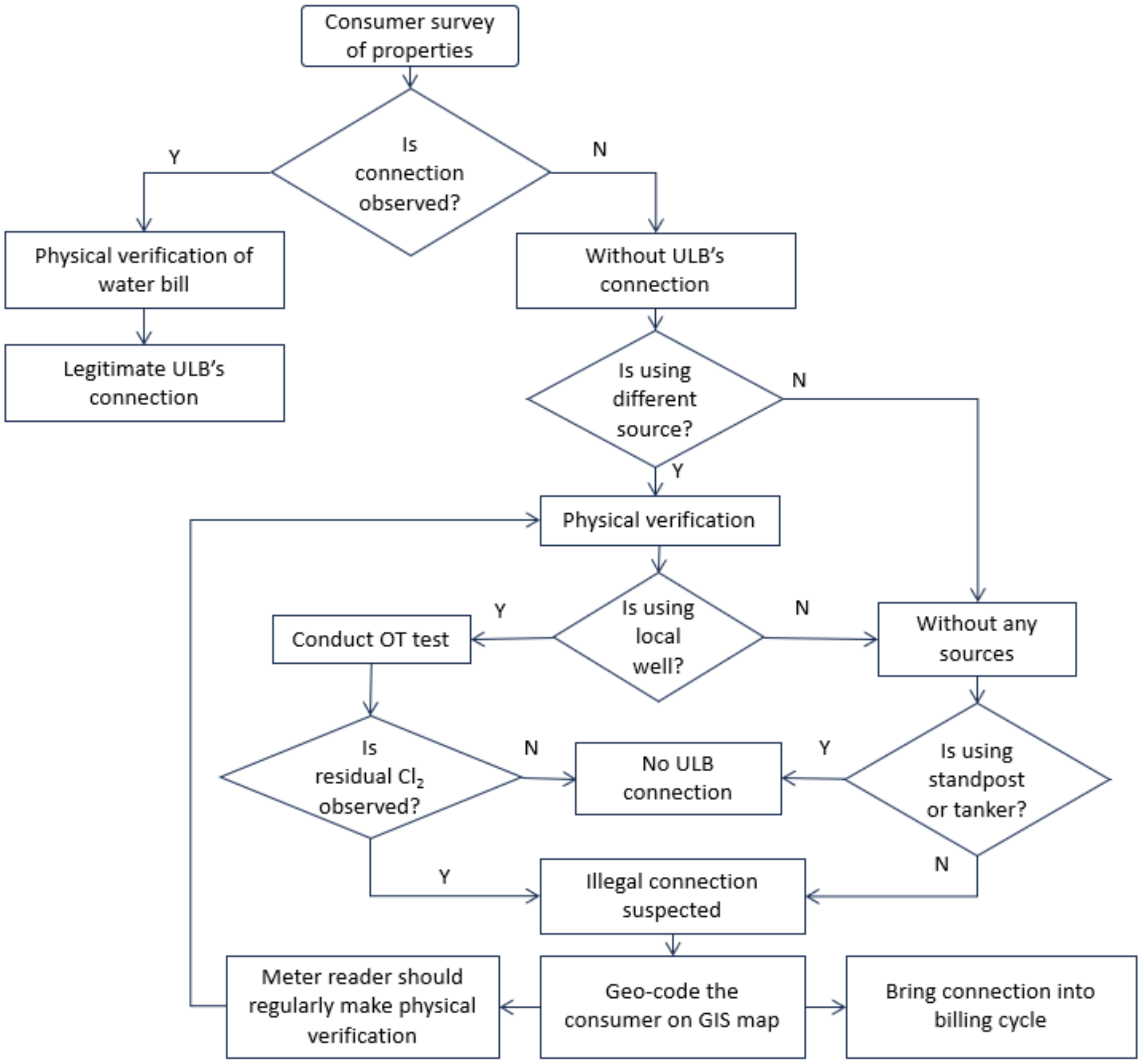


Figure 2.9: Steps of identifying suspected illegal connections

(iv) Replacing Old Pipes

While carrying out reconfiguration of network to isolate DMAs, replacement of heavy leaking old pipes, should be carried out. Old pipes having a previous record of number of repairs should be replaced. For each main line there is an economic range in which it is cost effective to carry out replacement. The process is explained in DMA management in section 12.16 of Part A Manual.

(v) GIS Mapping

GIS mapping is necessary and it has been discussed at length in the Guidelines of "GIS Mapping of Water Supply and Sewerage Infrastructure", released by the MoHUA in April 2020.

(vi) Customer's Underground (UG) Tank/ Sump

The UG tanks/ sumps are leaky and contaminated as per the study carried out at Nagpur by NEERI and VNIT, Nagpur by CPHEEO. Therefore, it is recommended that the buildings up to three storeys, there should be no underground tank/ sump at the customer's house to prevent seepage and contamination. If it exists, then after stabilisation of 24×7 pressurised supply, such tanks/ sumps shall be removed gradually in a phased manner. However, a building with more than three storeys can have watertight RCC underground tank with lining/ PE tank (with a maximum of two days storage at 135 LPCD). ULB's to develop a protocol (bylaws) for regular cleaning of such underground tanks.

The ULB shall monitor the monthly consumption of water for all households and check the households whose consumption is abnormal which may be due to leakage of water through seepage. The ULB should give warning to such households to repair/ replace their sumps either with RCC or PE tank. Till the 24x7 water supply is stabilised, all existing UG sumps which are constructed with brickwork have to be either plastered or converted to RCC or PE tank with storage capacity of two days. Once the 24x7 water supply is stabilised the UG tank may be delinked (for building up to three storey) gradually in a phased manner. If any household desires to create storage capacity even after getting 24x7 water supply to ensure water supply storage for emergency situation, it is recommended that households shall preferably create storage on the rooftop of their buildings with a capacity of 50% of their daily requirement as the distribution system is designed for residual pressures of 17 – 21 m and 12 -15 m as the case may be.

(vii) Strategy for increasing supply hours to 24 hours

The basic principle of conversion is to increase the supply hours of the existing system by saving water. This can be done by 100% consumer metering and management of demand by enforcing a telescopic (differential) tariff based on volumetric consumption. This means the more the consumption, the more is the tariff slab. Water can be saved by arresting the leakages in the system. Strategy of increasing supply hours to 24 hours is shown in Figure 2.10.

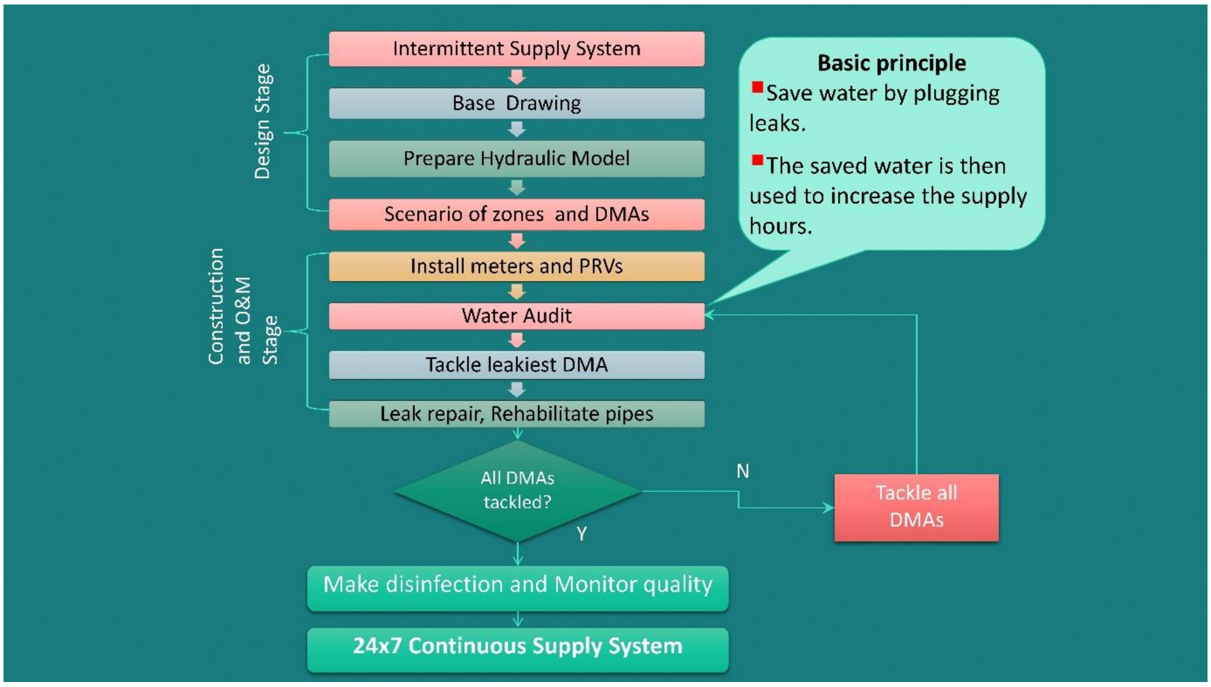


Figure 2.10: Strategy of increasing supply hours to 24 hours

(viii) Strategy for sustainable 24×7 pressurised system

Apart from the technical measures, a tariff strategy is required to save water by discontinuation of flat rates and charging on a volumetric basis by adopting telescopic tariff. Other measures such as organisational, commercial, policy, and budget are equally important. A summary of strategy for sustainable 24×7 pressurised system is shown in Figure 2.11.

All the above measures should be taken into consideration. If technical measures alone are taken, then the goal of conversion to 24×7 would not be achieved.

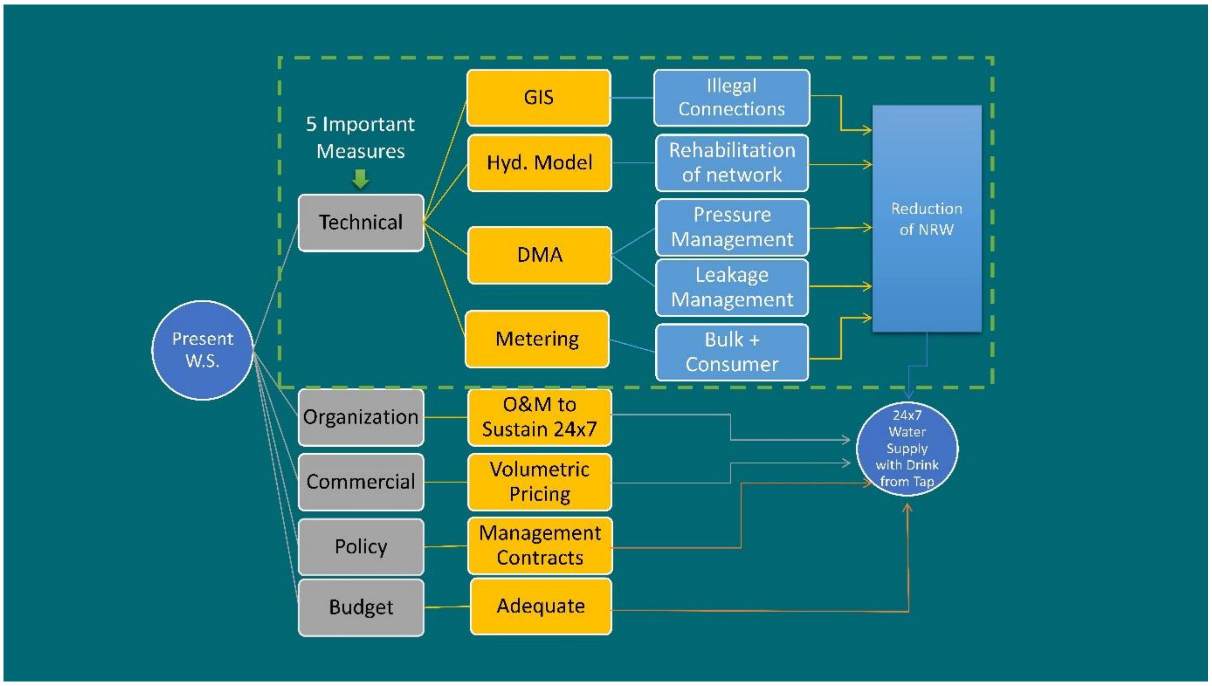


Figure 2.11: Strategy for sustainable 24×7 pressurised system

(ix) Disaster Management

Source and system failure in water supply can occur during disasters. When a source fails it takes a longer period for restoration of water supply. Therefore, there is a need for the preparation of action plans to mitigate disasters in the water supply systems.

Providing water supply in a disaster period is an important task for water supply authorities. There is a great risk of an outburst of epidemics if the water supply is not restored within a few hours of the onset of disaster. As disasters to water infrastructure cannot be clearly comprehended, it is very necessary to have perfect knowledge of the system.

If one source of the city is hampered, the system must ensure that it receives water from an alternate source to maintain a continuous water supply. The alternate source can be an unaffected source supplying water either to some other part of the same city or to other cities. Modelling the failure system is a critical part of designing and operating water networks so that the water system serves the community reliably, safely, and efficiently in the crisis period. Disaster management consists of the following phases:

Emergency Phases: General information on emergencies should be obtained. In routine operations of the water supply of the city, there may be some signs/indications seen before the actual outbreak of disasters. For example, during monsoon, daily rainfall data and river levels can give such warnings.

Apart from flood and loss of supply, contamination by chemical spills is also possible. Alerts based on change in water quality should be made available using appropriate technology. If water pollution is detected at an early stage, suitable measures can be taken so that critical situations can be averted. This can be done by doing water quality examination in real-time. Such smart solutions for monitoring of water quality are very important with advancement in sensors, communication, and Internet of Things (IoT) technology.

These measures will provide sufficient time to warn the consumers and implement mitigation measures designated to reduce loss of life and proper damage.

Some emergencies occur with little or no advance warning; for example, during the disaster of 26th July 2005, the heavy floods washed away the gates of the Badlapur barrage, which is a source of Ambernath and Badlapur cities, in District Thane. On the eventful day, there was historical heavy rains (940 mm in 24 hours). The gates were designed for earlier Highest Flood Level (HFL). But on that day, HGL was also changed and increased by 5 m. So, the gates were subjected with horizontal thrust of the flood water and were washed away. Pumping machineries were inundated. Rise in level was so rapid that 25 workers were trapped in the pump house.

Such type of incidents requires immediate activation of the emergency operations plan. All employees must be prepared to respond promptly and effectively to any probable emergency. Emergency management activities require the following phases:

Preparedness Phase: This phase involves activities taken in advance of an emergency. The hydraulic model simulating the operation of transmission mains and action plans should be prepared. Standard Operating Procedures (SOPs) should be prepared to respond to a disaster. It also involves a checklist mentioning staff assignments, notifications, procedures and resource lists. The maps of important valves should be shown on GIS maps and kept for display in the office of the city engineer and the building of WTP. The water works staff should be familiar with these SOPs and they should be trained accordingly. Apart from knowing where they are, they should be exercised on a regular basis so they can function during emergency situations.

Mitigation Phase: In this phase, besides the valve operations, actions should be taken to make regular water supply. For example, when the water level in the barrage is decreased due to the washing away of the gates, some pumps may be required to supply water in crisis. Mitigation should be thought of as taking actions to strengthen facilities and reduce the potential damage to structures.

A case study of source failure of water supply in the Mumbai metropolitan area is presented in **Annexure 2.9**.

(x) Activity Chart for Change of Mode from intermittent to 24×7 water supply

Common activities necessary for the adoption of the 24×7 water supply may be considered by the ULBs which are shown in Figure 2.12.

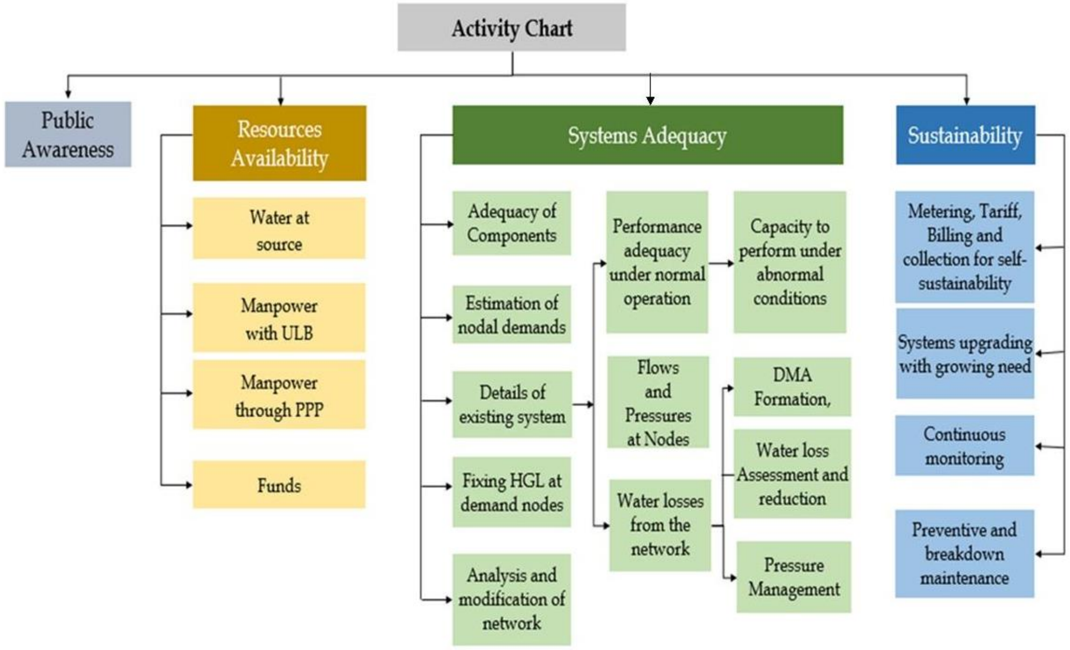


Figure 2.12: Activity chart showing a road map for change of mode

2.10.2 Implementation Steps for Gradual Conversion to 24×7 System

Detailed steps for gradual conversion through planning and implementation phases are as follows:

While planning the conversion process from existing intermittent system, it must be ensured that the residual nodal pressures in the existing OZ/DMAs shall be 17-21 m for Class I and II cities and 12-15 m for other cities. But in reality, it may be observed that the residual pressures are far less than 17-21 m as the projects in the past were designed with low residual pressures in the distribution system.

Hence, the first task is to achieve the recommended residual pressures in gradual manner. But the biggest challenge is that in most of the cities, the staging height of the service reservoirs is not enough, as a result the required residual pressures of 17-21 m could not be achieved. Hence, in the preparatory phase of planning and design, a strategy has to be evolved for achieving the required residual pressure. The detailed implementation steps for operationalisation of 24×7 system for the senior, middle and junior level engineers and the consultants are shown in Figure 2.13. Figure 2.13 is expanded in Figures 2.13 (a), (b).

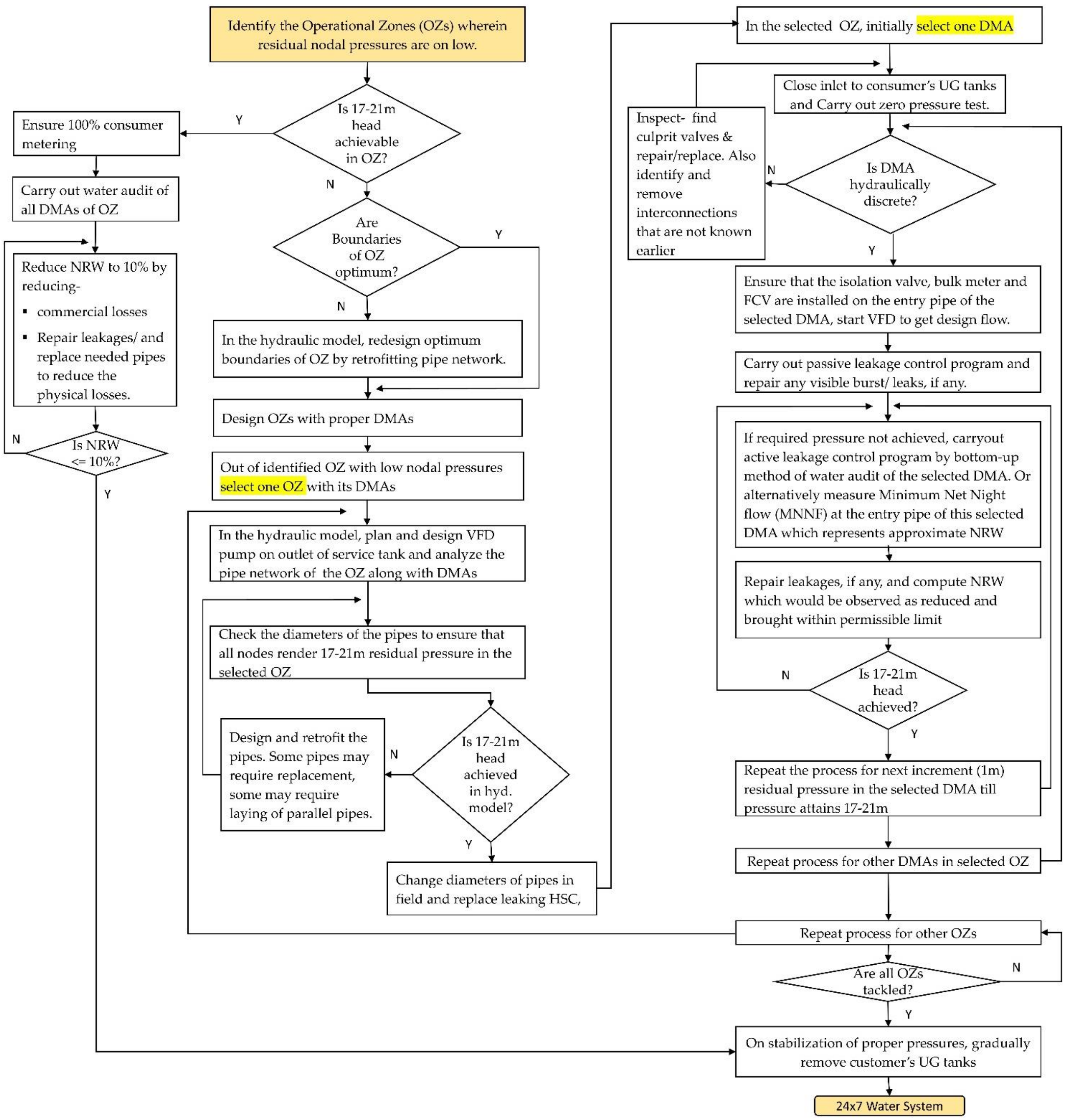


Figure 2.13: Detailed implementation steps for operationalisation of 24×7 Pressurised Water Supply System for the senior, middle, and junior level engineers and consultants.
[Parts of this figure are enlarged in Figures 2.13 (a) and (b)]

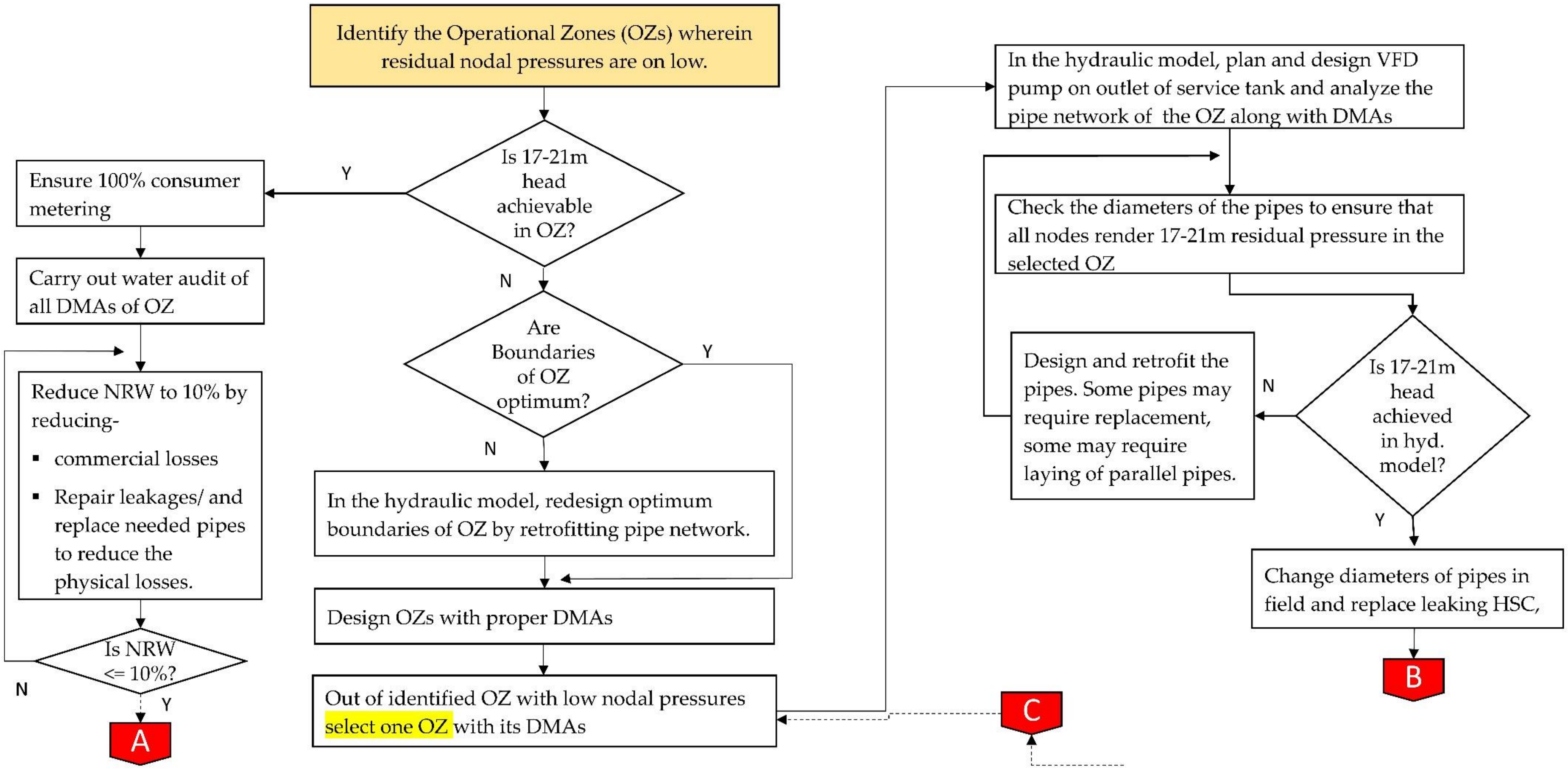

Identify the Operational Zones (OZs) wherein residual nodal pressures are on low.
Is 17-21m head achievable in OZ?
Y
N
Ensure 100% consumer metering
Carry out water audit of all DMAs of OZ
Reduce NRW to 10% by reducing-
▪ commercial losses
▪ Repair leakages/ and replace needed pipes to reduce the physical losses.
Is NRW <= 10%?
N
Y
A
Are Boundaries of OZ optimum?
Y
N
In the hydraulic model, redesign optimum boundaries of OZ by retrofitting pipe network.
Design OZs with proper DMAs
Out of identified OZ with low nodal pressures select one OZ with its DMAs
C
In the hydraulic model, plan and design VFD pump on outlet of service tank and analyze the pipe network of the OZ along with DMAs
Check the diameters of the pipes to ensure that all nodes render 17-21m residual pressure in the selected OZ
Is 17-21m head achieved in hyd. model?
N
Design and retrofit the pipes. Some pipes may require replacement, some may require laying of parallel pipes.
Y
Change diameters of pipes in field and replace leaking HSC,
B

Figure 2.13(a)

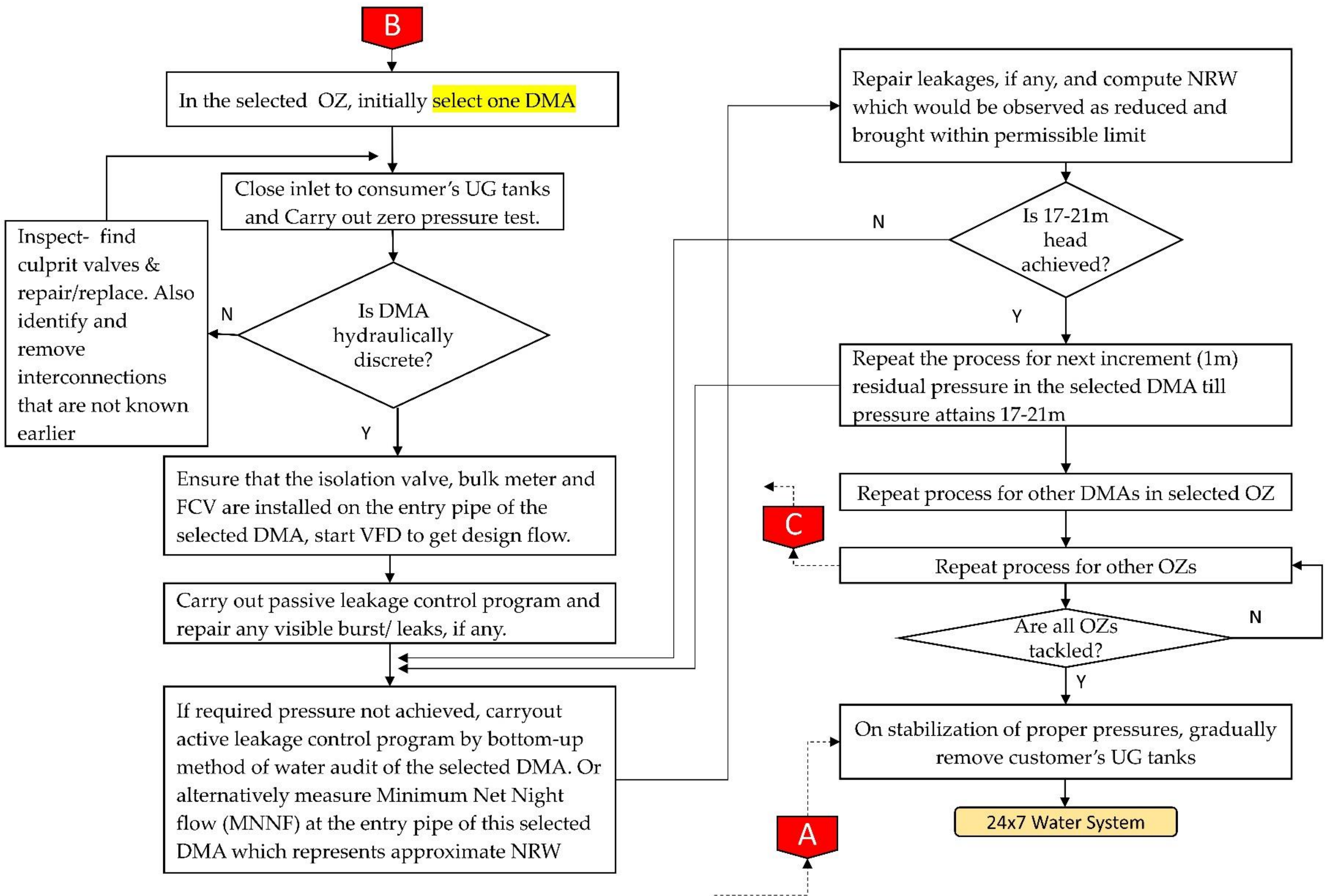

B
In the selected OZ, initially select one DMA
Close inlet to consumer's UG tanks and Carry out zero pressure test.
Is DMA hydraulically discrete?
N
Inspect- find culprit valves & repair/replace. Also identify and remove interconnections that are not known earlier
Y
Ensure that the isolation valve, bulk meter and FCV are installed on the entry pipe of the selected DMA, start VFD to get design flow.
Carry out passive leakage control program and repair any visible burst/ leaks, if any.
If required pressure not achieved, carryout active leakage control program by bottom-up method of water audit of the selected DMA. Or alternatively measure Minimum Net Night flow (MNNF) at the entry pipe of this selected DMA which represents approximate NRW
Repair leakages, if any, and compute NRW which would be observed as reduced and brought within permissible limit
Is 17-21m head achieved?
N
Y
Repeat the process for next increment (1m) residual pressure in the selected DMA till pressure attains 17-21m
Repeat process for other DMAs in selected OZ
C
Repeat process for other OZs
Are all OZs tackled?
N
Y
On stabilization of proper pressures, gradually remove customer's UG tanks
A
24x7 Water System

Figure 2.13(b)

Detailed steps in Figure 2.13 are explained as below:

(1) Dedicated NRW cell is required in each ULB which can take stock of situation and continuously monitor and reduce the NRW levels.

(2) Water quality cell is needed to continuously monitor and control the water quality.

(3) Identify the OZs in the GIS based hydraulic model wherein residual nodal measured pressures are low due to insufficient staging height of the ESRs.

(4) The residual nodal pressures are to be checked whether 17-21 m (or 12-15 m as the case) are be obtained or not. Henceforth, the required nodal pressures will be denoted by 17-21 m.

(5) If such pressures are obtainable as per hydraulic model, then ensure that DMA is made hydraulically discrete by closing boundary valves, then the 100% consumer metering should be done and DMA-wise, water audit in the OZ should be carried out. The NRW should be less than or equal to 10% in the DMA. If not, then we need to reduce NRW by taking NRW reduction programme. If the required NRW is achieved, then after stabilisation of the nodal pressures to 17-21 m, consumer UG tanks shall be gradually delinked. In this way, 24×7 pressurised supply can be achieved.

(6) However, as mentioned above in Sr. no. 4, if nodal pressure is less than 17-21 m, then the existing service tanks shall be studied whether they have optimum boundaries (proper allocation of command areas to ESR) or not. If not, the exercise of making optimum boundaries should be taken up in hand with the help of hydraulic model. This can be done by re-engineering and retrofitting the pipe network using the hydraulic model.

(7) On optimising boundary of the existing service tank, out of the OZs with optimised boundary, select one OZ with the lowest nodal pressure.

(8) In the hydraulic model, plan, and design the VFD pump on the outlet of the service tank and analyse the pipe network of the selected OZ along with DMAs for diameters of the pipes to ensure that all nodes would render 17-21 m residual pressure.

(9) Before checking the actual required field pressure of 17-21 m in the OZ, the capability of the OZ should be checked whether it is capable to create and sustain 17-21 m at the nodes. This should be checked using the hydraulic model. If the network is incapable to sustain the pressure, then design and propose retrofitting of the pipes using hydraulic model. Some pipes may require replacement with slightly higher diameters, some may require laying of parallel pipes.

(10) It is observed that about 70%-80% of the total leakages occur at ferrule point which is a start point of the HSC. In implementation stage such HSC shall be replaced.

(11) The change in network should be implemented on field and the nodal pressures shall be measured at the highest elevation of the DMA in the field. On achieving the required nodal pressure of 17-21 m, in the selected OZ, initially select one DMA.

(12) Carryout the zero-pressure test to ensure that the selected DMA is hydraulically discrete. Before conducting this test, ensure that the inflow to consumer underground (UG) tanks is closed by closing isolation valve and precaution shall be taken to see that the float is in good condition.

(13) If the test is negative (there is leakage), then inspect the circumferential boundary of DMAs where the isolation valves (if dead ends are not planned) are installed. Identify the culprit valves that are leaking. Either repair them or replace them. Also identify any interconnections between the adjoining DMAs that are not known earlier and plug them.

(14) On doing these actions again carry out the zero-pressure test. It should indicate that the DMA is 100% discrete (isolated).

(15) Ensure that the isolation valve, bulk meter and FCV are installed on the entry pipe of the selected DMA.

(16) Now that the arrangement is ready for performance observation, the flow in the DMA should be started slowly (up to design flow) by the installed designed VFD. The average observed nodal pressure shall be measured.

(17) Carry out passive leakage control programme. In passive leakage control programme, the visible burst/leaks are to be repaired. On removing such visible leaks/bursts, the residual nodal pressures are expected to increase.

(18) Still, if the required nodal pressures are not seen, the active leakage programme should be taken up in the selected DMA which is in the selected OZ. The active leakage programme can be carried out in three ways:

(a) If 100% consumer metering is done in the entire DMA along with isolation on the HSC, the procedure followed is to carryout leakage programme through bottom-up method of water audit (detailed in Part B Chapter 11: Water Audit and Leakage Control), in which the quantum of water coming in the DMA is measured by the bulk meter installed at the entry point of DMA and the total water consumption in DMA is measured by the consumer's meters. The difference of water coming in the DMA and water consumed in DMA gives the value of NRW.

(b) If 100% metering is not done in entire DMA, then the sub-DMAs are to be formed. At least 10% of the customers in the sub-DMA are to be metered. The inflow to the sub-DMA shall be measured by the portable flowmeter and the consumption shall be measured by meters in the sub-DMA.

(c) However, during the process of increasing nodal pressures to 17-21 m, quick determination of value of NRW is required. In the absence of formation of sub-DMAs and household meters, NRW can be approximately and quickly computed by measuring the minimum net night flow (MNNF) at the entry pipe of this selected DMA which represents approximate NRW. For this purpose, reading of the minimum night flow (MNF) should be taken from the bulk meter installed. Determine the legitimate night consumption such as consumption in hospitals etc. After deducting the legitimate night consumption from MNF, value of the net minimum night flow is measured, which indicates the approximate NRW in the network of the selected DMA.

(19) Identify the leakage spots while carrying out steps (a), (b), or (c) in the selected DMA and repair leakages, if any, and compute NRW which would be observed as reduced and brought within permissible limit.

(20) Repeat the process for the next increment of 1 m which is added to the average observed nodal pressure in the field at the highest node till the required residual pressures of 17-21 m are obtained in the selected DMA.

(21) Now repeat the above steps for all DMAs in the selected OZ. The NRW values in this selected OZ are expected to be reduced and the nodal pressures to the extent of 17-21 m are also expected.

(22) Repeat the above steps for the rest of the OZs.

(23) On stabilisation of pressures 17-21 m in all the OZs, the system is gradually converted into 24×7 pressurised water supply scheme.

(24) Now gradually delink consumer UG tanks by closing the valve leading to the UG tank and opening the valve on bypass arrangement for direct connection up to third floor. The delinking of UG tanks should be done through wider publicity.

(25) The water quality of all the OZs and DMAs should be sampled and monitored. If the required standard quality (as per BIS code IS10500:2012) is not met, then the corrective measures in WTP such as disinfectant's dose should be monitored. To assess quality in the distribution network, Orthotolidine (OT) test should be taken regularly, one sample for every 10,000 population once in a month. In addition to this, regular sampling and monitoring online or offline of pH and residual chlorine at farthest node of each DMA should be carried out and recorded for taking corrective measures if any.

2.10.3 Gradual increase in nodal pressure for cities

In the past, many water supply systems were designed for 7m or 12m residual head but operated with less than 7m or 12m due to field conditions and other reasons. In such a situation, if the staging height of service tank is sufficient enough to maintain the required pressure the following procedure shall be adopted.

Generally, isolation valve is installed on the outlet of service tank. This valve shall be opened very slowly with an increment of one thread at a time and then the residual nodal pressures in the OZ shall be checked. For this purpose, the pressure logger shall be installed at the critical nodes (highest elevation node). After opening the successive thread of isolation valve, the pressure at critical node is expected to increase. The NRW cell should inspect to check if there is any leakage in the OZ. After repairing such leak next operation of opening of the successive thread of the isolation valve shall be carried slowly and the process is repeated till we get required pressure at the critical node.

2.11 O&M phase (Phase 3)

2.11.1 Transition phase to operationalise 24×7 system

Even after implementation of the project, during operational stage, the value of NRW may increase continuously due to gradual increase of pressure during operation of VFD till the desired pressure is achieved. Therefore, the NRW control measure shall be continued while increasing the residual pressure to achieve the target residual nodal pressure of 17-21 m and reducing NRW to 10%. During this process water quality monitoring shall be continued to supply drinking water to every household free from biological contamination and meeting the drinking water quality standards of BIS (IS 10500:2012).

The continuous monitoring can be achieved by installing the SCADA/IoT system. The SCADA system generates a lot of data which is helpful. The generated data analytics and the predictive analysis is required and the same can be produced using digital twin technology.

2.11.2 Stabilising 24×7 Operation, NRW reduction and delinking of UG tanks

When 24×7 PWSS is commissioned, the residual nodal pressures are stabilised in all the nodes in the distribution network. With availability of 17-21 m the buildings up to three storeys need not have the underground (UG) storage tanks as these tanks leak and are contaminated due to entry of outside contaminants into it. This makes water non-potable. Therefore, after implementation of 24×7 pressurised system, the consumer's UG tanks should be gradually delinked. Initially, the consumers may not agree to do so. But when the 24×7 system is stabilised, the consumers shall get water continuously and they shall have confidence induced in the system. However, vigorous information, education, and communication (IEC) programme should be carried out by forming women's self-help group (Jalsathi's) like in Puri, Odisha.

House service connection pipe can be directly connected to the internal plumbing system so that can water can reach up to 3rd floor.

In case of high-rise buildings, the society (group of residence) may have the watertight UG tank constructed in RCC/PE. The water in UG tank may be pumped to their common overhead tank.

The UG tanks need timely cleaning operation at least once in six months. ULB's NRW cell can monitor this activity by conducting regular surveys.

2.12 Comprehensive Management Strategy

The management of water supply systems is the process of planning, developing and managing entire system from its source to consumer's tap so that the consumer gets adequate quantity of potable water. The management includes financial planning and management, monitoring and implementation of the project, structuring and implementation of differential water tariff to ensure sustainability, creation of enabling environment for Public-Private Partnership (PPP), capacity building, preparation of metering policy, asset management, stakeholder's engagement, MIS, O&M of water supply system implemented to achieve 24×7 pressurised system, monitoring of the SLBs, monitoring key performance indicators, continuous monitoring and reduction of NRW and the water quality monitoring and surveillance throughout the design period as detailed in different chapters of Part C of this manual.

The three phases, viz., Phase 1: Preparatory Phase, Phase 2: Implementation Phase, and Phase 3: O&M Phase need a very strong comprehensive management strategy from day one for successfully achieving and sustaining a 24×7 PWSS.

A comprehensive management strategy is very important and crucial for implementing the 24×7 PWSS and the phase-wise key management strategies are explained below:

THE STRATEGY

Phase 1: Preparatory Phase:

Survey and Investigations: Survey activities are the crucial building blocks for planning, designing, implementation and O&M of the existing and new system. The survey involves various activities which needs complete involvement of the authorities in facilitating the survey. This needs various permissions and information from different departments, e.g., ULBs water department, roads department, water resources department, forest department, railways and various government and private agencies. The condition assessment needs permissions to use various wireless instruments as well as digging the roads and diverting traffic with stopping water supply of certain section of network or facility. The survey may need use of drones which requires permission from the respective departments.

The consumer survey in preferably in local language is a very sensitive activity and needs an elaborated questionnaire with an access to visit the consumer premises. This will have to be facilitated by ULB officials with complete co-operation from the elected members for getting accurate consumer data for preparing an accurate network model as well as billing database. The accuracy of this data will be critical for successful implementation of 24x7 water supply projects to make the system financially attractive for any PPP Operator.

Once survey and investigation activities are over, the comprehensive data base should be prepared, maintained and uploaded in the ULB's web site so that it is made available to all the stakeholders.

Preparatory Phase - Planning and Design: The water supply systems are planned for a design period of 30 yrs. with 95% dependability. A sustainable source availability is critical and the ULB authorities have to work with water resources department authorities/groundwater development authorities to identify, survey, investigate and get permission for extraction of water at source. To develop the source from dams/reservoirs, permission and water reservation/allocation are needed from water resource department as well as forest department for construction of the intake structures as they generally fall under protected forest. The water lifting also needs approval and reservation/allocation with respect to the yearly quantity of raw water to be lifted.

Land is needed for all the components/structures of the water supply system, including, intake, approach roads, WTPs, pumping stations, ESRs and office premises. These permissions generally need serious interventions from all authorities and political fraternity at local, state and even national level for certain interstate sources. Sometimes the lands are owned by national organisations, defence or private owners which needs a clear land acquisition/transfer policy at all levels.

To supply affordable drinking water to every household as per BIS IS 10500:2012, it must be ensured that the selection of raw water source should not be contaminated with the discharge of industrial waste, hazardous waste, toxic waste and domestic sewage. It must also be ensured that the cities and towns receiving surface water in the downstream should take up with ULBs which are on the upstream and discharging municipal sewage and also other industries to adhere to the pollution control norms of the state and central authorities. The respective ULBs on downstream side may resolve the issues referring the issues to the respective state pollution board and also state government board. The state pollution control board and the industry departments will have to be taken into confidence.

Many times, the pipe alignments fall in the national/state highways/roads right of way or through forest areas and may need to cross railway lines also. These permissions need elaborated documentation and is time-consuming.

The city water balance plan has to be prepared by the ULB based on the concept of IUWRM to ensure water security throughout the design period as explained in section 4.13.

The population forecasting involves various departments e.g., Town and Country Planning Department, Statistics Department and Tourism Department (for floating population). While designing the project, the land use pattern, population growth pattern, population projection for a design period of 30 years shall be finalised in consultation with Town and Country Planning Department of State Government, wherever necessary.

The states must have a legal and institutional framework (as discussed in Chapter 2 of the Part C of the manual) in place at state and ULB level, which forms various policies, issues advisories, initiate various investment programmes as well as data and information transfer initiatives in the sector. There is also a strong need for regulation in the urban water sector. The water policies, including tariff setting, have to be framed and implemented by State/ULB at the planning stage itself for implementing 24×7 PWSS which is technically and financial sustainable. These issues have been discussed and addressed in various chapters of Part C - Management, of this Manual.

Phase 2: Implementation Phase:

Prerequisite: During the implementation phase, various activities like removing public stand posts, identification and replacement of faulty HSC, old pipes, pumping machinery, regularisation of Illegal connections, identification and planning of the construction of new WTPs and ESRs, SCADA, instrumentation, establishment of water quality laboratories, etc., will have to be carried with the legal framework, institutional staff arrangements and stakeholders engagement with active involvement of the ULBs. NRW cell and water quality monitoring cell shall be established in ULB.

ULBs should initiate action to formulate their own metering policy, tariff policy and connection policy as per the respective model policies provided in Chapter 13 of Part A of this manual.

Capital works for Gradual Conversion to 24×7 PWSS and New System - Implementation Steps: The conversion to 24×7 project involves preparation of DPR which includes all the capital works, O&M costs, project development costs along with the land acquisition. The costs for power supply and environmental, social and gender safeguards should also be included. The funding of the project will need strong financial systems in place and efficient billing and collection. Funding from state, central and multilateral agencies will have to be studied and a funding strategy has to be put in place. The cash flow to maintain the funds for execution of works has to be embedded in the budget of the ULBs. ULBs should ensure that 100% consumer metering with incremental differential (telescopic) tariff including subsidy for urban poor based on volumetric consumption for 30 years to sustain O&M cost. PPP option has to be explored with a detailed study of the suitability of the PPP model so as to attract private agencies. All above including the PPP part is covered in Part C, Chapter 8 - Public Private Partnership of this Manual. This has been explained in Part C, Chapter 4 - Financial Management of this Manual.

In the Guidelines for AMRUT 2.0, it is mentioned that projects on 24x7 pressurised water supply system with drink from tap facility may be taken up.

However, in order to ensure speedy implementation of 24x7 PWSS project, the city needs to prioritise the implementation of various project components in a phased manner. In this regard, it is recommended that the cities should initially implement water distribution network in the project area or the whole city by considering OZs and DMAs with inlet and outlet arrangements (bulk flow meters, isolation valves, pressure valves, HSC up to boundary of the premises etc.) to facilitate better utilization of the capital investment available under time bound missions like AMRUT 2.0 or State Funds. Immediately after the formation of all OZs and DMAs, the cities shall initiate action to connect the house service connections with houses along with water meters for gradually achieving 24x7 PWSS in one after another DMA and upscale to project area or entire city in a phased manner as clubbing the laying of main distribution network and providing house service connection with meters simultaneously will delay the commissioning of the overall project.

After completing the replacement of pipelines and HSCs in DMAs, ULB should initiate action to undertake NRW reduction programme and monitor the same using various modern metering and communication methods suiting to respective cities and towns as discussed in Chapters 13 and 14 of Part A of the manual.

It must be ensured that water quality monitoring and surveillance should be undertaken as per the guidelines given in Chapter 8 in Part B of this manual.

Considering the climate change impact on the water availability, utmost care has to be taken to design the component of works which are climate resilient. This aspect has been discussed in Chapter 9 - Building Resilience for Climate Change and Disaster Management in Part C of this Manual.

Phase 3: O&M Phase:

It is necessary to make timely daily operation of various components of the water supply system such as headworks, treatment plant, machinery and equipment, transmission mains, service reservoirs and distribution system, etc. The operation of 24×7 PWSS should be done in efficient and economically way, so that the aim of supplying safe and clean water in equitable manner to the consumers is achieved.

It is needed to maintain water supply system efficiently. Maintenance is an art of keeping the structures, plants, machinery and equipment and other facilities in an optimum working order to attain proper functioning without any interruption. Maintenance is of two types - preventive maintenance and corrective maintenance. All aspects of O&M are discussed in Part B of this manual.

Transition Phase to Operationalise 24×7 Pressurised Water Supply System Including NRW Reduction and Monitoring Water Quality: During the transition phase to operationalise the 24×7 system, more emphasis will have to be given on the DMA management and data collection. The stakeholder's engagement is going to play a crucial role in making people accept metering, their willingness to pay for good services and good water quality by implementing 24×7 PWSS with DFT. The assets installed, e.g., pipes, meters, etc., have to be managed by good asset management systems so as to monitor the transition activities. Institutional strengthening is essential to have trained and efficient staff to carry out all the transition activities and operate 24×7 PWSS. The self-help group, for example, Jalsathi's in Puri, NGOs, residential welfare associations, etc., will play an active role in this phase. These issues have been discussed and addressed in Chapter 3: Institutional Strengthening and Capacity Building of Part C Manual.

Stabilising 24×7 Operation, NRW Reduction and Gradual Delinking of Customer's UG Tanks and Monitoring Water Quality Continuously: Stabilisation of the system will increase the confidence of the people in the water supply system and the ULBs will be in a position to delink the underground (UG) tanks through vide publicity and achieve consumer satisfaction. This will also increase the revenue of the ULB/PPP operator, thus achieving financial sustainability, which ultimately increase the quality of life of the people. Continuous monitoring via. MIS and regular stakeholder engagements will make the system efficient and robust. Efficient O&M with strict Water Quality Monitoring will be the key for sustaining the success of the project with DFT mission. The O&M activities, including Water Quality Monitoring and Surveillance has been explained in Part B - O&M, of this Manual. The Management Practices can be referred in Part C - Management of this Manual.

Capacity Building

Capacity building is paramount important to operate and maintain the 24×7 PWSS throughout the design period as ULB requires skilled manpower. It must be ensured that the engineers of ULBs and that of the state departments should be trained through various central and state government PHE training programmes as discussed in Chapter 3 of the Part C of this manual.

Since many ULBs lack technical capacity to plan design, implement and operate maintain and sustain 24×7 PWSS, ULBs are encouraged to implement, operate and maintain water supply system through PPP mode on long term basis as discussed in Chapter 8 of the Part C of the manual.

Reforms in Governance for O&M of Water Supply Systems

Urban Local Governments were empowered through the 74th Constitutional Amendment Act (CAA) in 1992 to undertake 18 functions including water supply and sanitation services as per the 12th Schedule in the Constitution which contains the power, authority and responsibilities of Municipalities. But despite three decades of empowering ULBs through 74th Amendment to the Indian Constitution, India's Local Government still requires many administrative and financial reforms apart from technological and capacity building reforms.

As per the constitutional amendment, ULBs are mandated to oversee the planning, implementation and O&M of water supply systems. Still, the current practice of project implementation is done by the State PHE Department, Boards etc. and ULBS are responsible for O&M of the completed project through ownership transfer from State PHEDs to ULBs. This practice has not been yielding the desired optimum management of service delivery system. This issue needs to be addressed so that agency who is implementing the project shall also operate and maintain the system.

Henceforth, the future water supply projects are to be planned, designed, implemented, operated and maintained to provide 24×7 PWSS with an objective to supply water up to consumer end as per BIS (IS 10500:2012). It is of utmost importance that the scheme implemented by the State PHEDs and Water Boards should be operated and maintained by the same agency in order to ensure successful operation of 24×7 PWSS as envisaged during project planning and sustain the services throughout the design period by undertaking various measures including monitoring of NRW reduction, water quality and the service levels. Therefore, following reform measures are needed in all the States and UTs for effective planning, design, implementation and O&M of 24×7 pressurised water supply projects in a sustainable manner:

i. PHE Departments, individually headed by Pr. Secretary and the Municipal Administration Departments headed by Pr. Secretary, be brought under one umbrella of administration headed by the Additional Chief Secretary level officer.
ii. Intertwining the implementation and operation of water supply and sanitation project to share the knowledge of infrastructure design, implementation and their operational management aspects.
iii. Ownership building at different level of operational training by bridging the gap between silo approach of construction and operational activities with no system transfer at any level and instead, a common pool of officers (like state public health engineering services) at all required levels drawn from both the streams without losing their own cadre, be engaged and made jointly responsible for effective water supply and sanitation service delivery system as encompassed under the 74th CAA 1992.

2.13 Summary of Planning and design norms

The design norms for the capital works are summarised in Table 2.7 and for sustainable O&M of continuous (24×7) water supply systems in Table 2.8.

2.14 Dual Water Distribution System (DWDS) in Coastal Cities

Dual water supply systems consist of two independent pipe networks with separate treatment, pumping and storage system to supply different grades of water to consumers for potable and non-potable applications. DWDS may be planned and designed in the following two cases:

2.14.1 Case 1: Coastal Cities and Towns

Most of the coastal cities & towns face the problems of saline water intrusion, thereby increasing the TDS in ground water not rendering the water for domestic consumption. Further, fresh water from either the surface water or distant ground water sources is available in limited quantity. In such cases, the coastal cities are forced to adopt desalination plants to meet out their fresh water demands. The capital and O&M cost of desalination plants with raw water source either from sea water or brackish water is very high and therefore such cities/towns shall explore the possibility of adopting dual water distribution system, where one pipe will convey limited quantity of potable water/desalinated product water, say minimum of 40 LPCD with peak factor of 2 for potable uses like drinking, cooking and bathing as piped water supply below this rate may have operation problems; and another pipe will carry water with high TDS saline ground water (not sea water) that is acceptable by community for toilet flushing and other uses with peak factor of 2.5. This option may be economical as compared to desalination plants and shall be considered by coastal cities/towns. The existing distribution system shall be retained to supply water for other purposes.

It must be ensured that the first pipe should carry 40 LPCD of water with low TDS, preferably less than acceptable limit of 500 mg/L or relaxed TDS value as decided by the competent authority as per the field conditions, i.e., Chief Engineer of the State/UT Govts. and another pipe should carry water with TDS not more than permissible value of 2000 mg/L for other uses such as toilet flushing, washing of cloths etc. High TDS water affects the metallic pipes and plumbing fixtures and reduces their lifespans. Therefore, HDPE and O-PVC pipes are more suitable for conveyance of high TDS water.

The city should carry out the techno-economic feasibility to adopt DWDS for supply of dual quality water *vis-à-vis* desalination treatment plant to meet the additional water requirement with conventional single pipe system.

The Dual pipeline carrying 40 LPCD should be designed and operated with 24x7 pressurised water supply system to prevent entry of outside dirt/wastewater in the pipeline during non-supply hours. Operationalising 24x7 pressurised system with 40 LPCD will be great challenge and it requires skilled manpower. However, the decision whether to adopt dual piping system or Desalination plant (to meet partial or full demand) is completely left with State Govt/ULBs/Parastatals.

The rationing of potable water is essential to ensure equitable distribution of water to all households, various commercial establishments and institutions and the required quantity of water can be restricted by installing flow meter with solenoid valve.

2.14.2 Case 2: Water Scarce Areas

Recycling and reuse of tertiary treated water in residential, commercial and industrial complexes at local level is being practiced in many cities to reduce the freshwater requirement. For example, Nagpur Municipal Corporation (NMC) is supplying 200 MLD of tertiary treated water to one of the power plants; Bangalore Water Supply and Sewerage Board (BWSSB) is supplying 4 MLD tertiary treated water to Vidhana Soudha, Raj Bhavan, Legislators home, Cubbon park and other important areas in central Bangalore from last 10 years for non-potable use; Nada Prabhu Kempe Gowda Layout (NPKGL) developed by Bangalore Development Authority (BDA) has planned and implementing to supply tertiary treated waste water for non-potable purposes with a dual water supply network. IISc Bangalore campus is supplying 1 MLD (whose requirement of fresh water is around 4 MLD) of tertiary treated water using MBR technology for gardening, cooling, toilet flushing etc. with a dual water supply system from last 7 years. However, Dual Water Distribution System need not be used in the part of cities and towns where water supply is already provided and because the households may not be willing to convert their plumbing system to dual plumbing system to supply potable water for drinking & bathing from one pipe and tertiary treated water for toilet flushing from another. Therefore, dual water distribution systems are recommended only in new layouts particularly in water scarcity towns so that one pipe will carry potable water for potable use and another will carry tertiary treated water for non-potable use such as toilet flushing etc. subject to the condition that the households in the new layout agree to adopt dual plumbing system in their respective houses/flats.

In the dual water supply system - two separate pipelines are to be provided clearly demarcated with different colour coding - one for potable water supply distribution to consumers ferrule through *blue colour* lining on pipe and other for supply of recycled treated wastewater to house flushing through *brown colour* lining on pipe. Accordingly, the consumers will be required to have dual plumbing system network within the households/premises with blue and brown colours lining on two separate piping system - one for potable water supply faucets/taps and other for flushing system.

"National Framework on safe Reuse of treated water in urban India" published in November 2022 by Namami Gange may be referred. The norms provided by CPHEEO for recycling and reuse of water for various specific purposes including toilet may be referred to at the Ministry website (https://mohua.gov.in/). Also, the BIS (IS 17663: 2021) which provides guidelines for water reuse safety evaluation- assessment parameters and methods for water reuse in urban areas may be followed for regular quality monitoring.

States and ULBs shall also encourage recycling of wastewater for non-potable applications within the premises of the large size residential apartments/Individual Households and commercial establishment to conserve fresh water.

A minimum diameter of 63 mm is recommended for dual piping system in case 1 and 2. However, the minimum diameter may be relaxed as per the field conditions. The city should carry out the techno-economic feasibility to adopt DWSS for supplying fresh water as well as tertiary treated water in coastal areas and water scarce areas.

Table 2.7: Recommended norms for planning, design and implementation- Capital works

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
1	**Design period (refer table 2.2)**	(a) Headwork should be designed for 50 years. (b) Units for Intermediate Stage: Tube wells/ bore wells, WTPs, CWRs and pumping machinery should be designed for intermediate stage and land should be kept available for ultimate stage and for future expansion. (c) Ultimate stage: ESRs and all pipelines including raw and treated water transmission mains, distribution pipes, pump house.	Base year: means proposed date of completion of the scheme. Intermediate: is computed as base year +15 years. Ultimate stage: is computed as base year +30 years.
2	**Land required for water supply infrastructure**	City planners should earmark the land required for water supply infrastructure and its expansion of ultimate stage in the master plan of the city for next 30 years or more.	Land is required for WTPs, sumps, ESRs, etc. When land for water supply infrastructure and its expansion is not available, the city planners may earmark in recreational amenities or parks, stadium, etc.
3	**Population forecast: Ward-wise forecast of population and population density**	Not only total population of city but its ward-wise distribution and computation of ward-wise future population density based on equivalent area is necessary.	This (nodal demand by future population density) has been discussed in **Annexure 2.7** along with the case study.

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
4	**Per capita supply of domestic/non-domestic for design (refer table 2.4)**	Cities/ towns with population less than 10 lakhs should be 135 LPCD Larger cities having population of 10 lakh or more should be designed for 150 LPCD. Non-domestic demand, bulk supply, etc., should be assessed as per the actual consumer survey. The non-domestic demand should be assigned to the respective nearby nodes. Fire demand should be added to domestic demand proportionately.	Supply should be at the consumer end. This means physical losses should be added to the demand. 1. The Metro and Mega cities should plan for water supply projects considering a per capita water supply of 150 LPCD and should take up underground sewerage systems within three years of commissioning of water supply scheme. 2. The other towns which are planning for water supply projects considering 135 LPCD should also take up undergoing sewerage system within three years from the commissioning of water supply scheme. 3. In case towns are facing water scarcity and are not contemplating sewerage system in the next 5 years, they can restrict per capita water supply to 100 LPCD for water supply projects and plan for decentralised sewerage facilities with on-site

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
			system as recommended in Sewerage Manual.
5	**Floating population**	Rate of supply for floating population should be as follows: i) Bathing facilities provided: 45 LPCD ii) Bathing facilities not provided: 25 LPCD iii) Floating population using only public facilities (such as market traders, hawkers, non-residential tourists, picnic spots, religious tourists etc.): 15 LPCD.	Figures should be got certified by ULB/ Tourism Department/ Statistical Department.
6	**Total demand**	The domestic demand does not include bulk requirements of water for semi-commercial, commercial, institutional, and industrial. Demands due to commercial, institutional, and industrial must be assessed separately through consumer survey and duly extrapolated for different stages. In the absence of consumer survey, the present demand due to semi-commercial to the tune of about 5%-10% of intermediate demand (domestic) may be considered depending on the nature of the town. The semi-commercial demand for intermediate and ultimate stages may be calculated considering an	Consumer survey of the city is mandatory for commercial, institutional, and industrial establishments (such locations can be easily identified using Google Earth). Consumer survey helps to ascertain requirement of consumer meters, identifying suspected illegal connections and for shifting of connections from main line. After deciding these values of demands, hydraulic modelling

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		increase of 1% per year on the initial semi-commercial demand. Fire demand should be added to domestic demand proportionately. Total demand should not exceed 15% and should be computed by adding following indicative losses: • Headwork to the inlet of WTP should not be more than 1% • In WTP, losses should not be more than 3% • Outlet of WTP to Various ESRs losses should not be more than 1% Sometimes, the location of WTP is close to headwork, and sometimes it is close to the city boundary. Hence, (a) and (c) above put together shall not be more than 2%. However, if (a) and (c) together is more than 20 km, then total loss should be considered at the rate of 1% per 10 km, instead of 2%. In a distribution system, losses should not be more than 10%. (With 24×7 project with 100 % metering NRW is expected to be reduced. Hence total losses in the distribution shall not be > 10%).	(design of distribution system) should be taken up.

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		For ground water (with appropriate treatment) where water is directly supplied to distribution system and WTP is not part of the system, the total loss should not exceed 11%.	
7	**Supply Hours and Peak Factor**	(a) The transmission system for both raw water and treated water including all pipelines up to ESRs should be designed for 22 hours of supply. (b) Water distribution networks of urban schemes: Peak factor should be designed for a peak factor of 2.5 irrespective of population. (c) Water distribution networks of rural part of urban-rural schemes: A peak factor of 3 irrespective of population should be adopted in rural areas.	On stabilisation of the water supply systems, peak factor may reach to the optimum value, based on the internationally established 24×7 water supply system.
8	**Minimum Diameter of Pipe for water distribution**	Minimum of 100 mm for all primary pipes in all the cities (for new pipes). In case the existing pipe is 80mm, the same may be retained in the system. In hilly terrain, 80 mm can be considered as the minimum size of pipe (for new pipes) for primary pipes. In case the existing pipe is 63 mm, the same may be retained in the system. For secondary pipes in small lanes of hilly areas for facilitating with the HSC pipes, the diameter shall be between 32-63 mm as per the local conditions.	

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
9	**Public stand post**	No new stand post should be given. Existing stand posts should be removed and converted to house connections with meter by formulating OZ-wise time bound programme by ULB.	Metered tap connections to all households are necessary.
10	**Minimum residual head at ferrule**	The residual nodal pressures at ferrule at highest node shall be 17-21 m for Class I and II cities and 12-15 m for other cities. **For existing ESRs:** In case staging height of existing ESR is not sufficient to develop designed residual pressure of 17-21 m or 12-15m as the case may be, the size of OZ shall be restricted based on the capacity of ESR (ultimate stage population). The VFD shall be designed taking into account the positive suction head (potential energy due to staging height). However, it is to be ensured that water level in the service tank should be maintained and the VFD pump shall automatically stop with dry running condition. If necessary, bypass arrangement may be made between inlet pipe and outlet pipe. The operation of the VFD pump shall be regulated through smart solutions by installing sensors at critical node of the OZ/DMA.	Though earlier manual (1999) recommended 7 m for single storey, 12 m for two storeys, 17 m for three storeys, and 22 m for four storeys, in practice, most of the cities have designed and implemented their projects with residual pressure of 7 m or 12 m irrespective of whether the cities have two or three-storeyed buildings. Because of this, water supply systems have to resort to the consumer's underground tanks. In a recent study conducted by CPHEEO through VNIT and NEERI, Nagpur on water quality deterioration and water quantity loss through seepages from consumer's underground sumps in the DMA of Nagpur city where 24×7 water supply is provided, it was observed that:

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		New ESRs: All new ESRs has to be constructed to maintain residual pressure of 17-21 m or 12-15 m as the case may be.	a) 42% of samples (25 number of sumps out of 60 total number of samples) had presence of indicator bacteria E-Coli/Thermotolerant Coliforms in the sumps. However, only 5% samples at inlet to the sumps were having presence of E-Coli. It means that the underground tanks are contaminated by seepages from outside contaminants. b) Number of samples from sumps having free chlorine less than 0.2 mg/L were 35%, while the samples from inlet having free chlorine less than the 0.2 mg/L were 10% only. c) 12% of the consumer sumps were observed leaking significantly. The quantity of water loss was observed varying from 13.20% of total household demand to as high as 223% as that of total household demand with an average of 98.27% of total consumer demand. Thus, the total water loss was 15.95 KL as against the total supply of 29.45 KL

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
			calculated based on 150 LPCD from seven households. In old areas of city, despite pipe material being metallic, many times the joints are weak due to aging specials of jointing of pipes. Even in such situations, pressure should not be relaxed. A systematic pipe replacement programme may be carried out stage wise in such cases.
11	**Maximum staging height of ESR**	Maximum staging height may be proposed to meet the residual head of 17- 21m.	To achieve above minimum head of 21 m and to have optimum velocity to achieve economical design of all pipelines in distribution, the staging height of the new service reservoirs should be appropriately chosen.
12	**Capacity of ESRs/ GSRs**	Balancing capacity of the service reservoir shall be calculated by: (i) mass balance, or (ii) 33% of the total demand of ultimate stage (30 years from the base year) of the OZ of that ESR. In any case, the minimum capacity shall not be less than 33% of the demand as above.	In case the VFD pumps are adopted for direct feeding the network, the sump acts as a service reservoir and provision of capacity mentioned in Col. 3 applies to this as well. Side Water Depth (SWD) if excessively chosen then the ESRs do

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		However, for rural areas the service tank may be designed for 50% of the ultimate demand.	not work efficiently. The maximum SWD should be as under: • For ESR capacity up to 1 Lakh litres: 3 m • For ESR capacity up to 10 Lakh litres: 4 m • For ESR capacity > 10 Lakh litres: 5 m
13	**Fire demand**	Prior to computation of fire requirements of OZ, it is necessary to compute the fire requirements for the entire city using following formula: For cities with population more than 50,000: $\text{Fire requirement for entire city} = 100\sqrt{P}$ (m^3/day) Where P is the intermediate stage (15 years) population of the entire city in thousands. $\text{Fire requirement of OZ} = \left(\frac{\text{Intermediate population of OZ}}{\text{Intermediate population of the entire city}}\right) \times (\text{Fire requirement of the entire city.})$	It is desirable that one-third of the firefighting requirements of each OZ form part of the service storage. For this purpose, the outlet of the tank supplying water for normal operation should be kept just above this storage so that the capacity provided for mitigating fire is always available. There should be fire outlet at the bottom of the tank that can be opened when an instance of fire occurs as well as at the time of cleaning the tank. The balance requirements maybe met out from secondary sources. The high-rise buildings should be provided

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		In case the service reservoir is designed for ultimate stage the word “intermediate” shall be replaced by Ultimate in above formula. For cities with population less than 50,000: Fire demand of OZ shall be computed initially for 50,000 and then proportionately decreased accordingly.	with adequate fire storage from the protected water supply distribution. Also, there is a remote possibility that the fire occurs at multiple places, hence nearby ESRs can also be used for firefighting requirement. The location of fire hydrants should be decided in consultation with fire department. However, arrangements for filling vehicles of fire brigade should be provided at each ESR. The pressure required for firefighting would have to be boosted by the fire engines.
14	**GIS Mapping**	GIS mapping of all the existing, proposed and executed infrastructure is required. GIS maps of ward boundary should be adopted for estimating demand by future ward-wise population density method. Training courses on GIS should be organised for capacity building of ULB’s engineers and planners.	

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
15	**Consumer meters**	Distributing water with 100% consumer metering is most essential. Hence, consumer metering is necessary. Details of metering policy are mentioned in section 13.2 of Part A of this manual.	Demand management is not possible in case of unmetered water supply at flat rate. Therefore, policy should be adopted for 100% house metered connection by the ULBs. Geo coding with GIS coordinates of all the consumer and bulk meters is mandatory.
16	**Water tariff**	Volumetric telescopic tariff structure is mandatory. This method, will help to supply water to urban poor at affordable price, encourage consumers to decrease their consumption and penalise for their excessive consumption.	It is required for controlling demand and hence it is an important tool for demand management. 100% household are to be supplied water through house metered connection (without public stand posts), first slab of telescopic tariff structure should be such designed that the urban poor can get drinking water at affordable price. Quantum of subsequent slab should be so designed that the middle-class persons get incentive for decreasing their consumption. At the same time, this slab should not be too costly to poor to maintain minimum hygiene

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
			standards. Quantum of subsequent slab/slabs for higher consumption shall be such priced that it becomes penalty for excessive consumption.
17	**Hydraulic Modelling**	Hydraulic modelling is required for planning and designing OZs and DMAs required for 24×7 water supply system. GIS based hydraulic model should be adopted which is effective in O&M. Values of elevations and demands must be given to each node using GIS and the software tools. Only two hydraulic models should be prepared for entire city - (i) for entire distribution system and (ii) for raw/treated transmission mains. If the city is exceptionally large and is divided into big zones, then the two models as above should be prepared each for the respective very big zone.	Hydraulic model should not be prepared in pieces. If it is done in pieces, the contours will not be seamless. In such case proper elevations should be assigned to the nodes. And the nodes will have incorrect elevations, and this will vitiate the hydraulics of the network. The water demand on nodes shall also be rationally distributed. The assignment of ground elevations and nodal demands to all the nodes in city should be given, i.e., to follow “whole to the part” method and not by the “part to the whole” method. Hydraulic modelling can be done using various software including freeware available in public domain.
18	**Creation of OZ**	The main principle of decentralised planning is that each service reservoir should have one OZ. These	OZ boundary is determined with help of natural features like the roads,

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		OZs are further sub divided in DMAs. Each OZ and each DMA should be hydraulically discrete. Such OZs should be created for entire city by following proposed hydraulic parameters of residual head and the respective peak factor.	railway line, nalla etc. and slope within OZ area. Normally in non-hilly area the slope within OZ should be up to 5 m. In case of direct pumping, pressure zones shall be formed using the GIS technology and then the number of OZs shall be computed. The transmission/feeder mains shall be so designed that all the OZs should be brought on a co-ordinated sharing in case of a massive disruption in one OZ, it should be possible to make up the restoration from other zones.
19	**Optimised boundaries of OZs**	If the extent of OZ is not sized, designed, and maintained properly, it leads to malfunctioning of storage reservoirs like emptying and overflowing. Hence, boundaries of OZs should be optimised.	In the current (existing) systems, optimum boundaries of OZ are not designed scientifically hence this exercise should be made as described in section 12.11 in Part A of this manual.
20	**Maximum size of OZ**	The size of OZ for new service tank should not be more than 50,000 population or 10,000 connections.	Oversize OZ will be difficult to operate and maintain, i.e., to provide equitable

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		For hilly areas, maximum ultimate population per OZ should be 30,000 or 6,000 connections. For size of OZ for existing service tank should be based on capacity of the existing service tank which will meet the demand of ultimate stage. In saturated/high density population areas where land is a constraint construction of service reservoir for catering OZ with 50,000 population, the norm of 50,000 population per OZ shall be relaxed and ultimate population up to 75,000 to 100,000 shall be considered in OZ with proper justification. However, maximum no. of household connections shall be restricted to 3000 by increasing the suitable no. of DMAs.	distribution of water and designed residual head and, hence, its size be limited.
21	**Design of DMA, its boundary, and Maximum size**	Number of DMAs in one OZ should not be more than four but preferably two or three and each DMA should be hydraulically discrete. Each DMA should have HSCs in the range of 500 to 3000 in plain areas and 300-1500 in hilly areas for ultimate stage. The size of an individual DMA may vary, depending on number of local factors and system characteristics. All DMAs should be fed by common pipe from outlet of ESR in OZ with branches and from these	OZ and DMA boundary is determined with help of natural features like the roads, railway line, water bodies, nalla etc. and slope within OZ area. For newly proposed tank, there should be separate outlets from the tank for each DMA.

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		pipelines, consumer connections should not be given. Each DMA should have only one inlet. By this arrangement and by limiting the size and boundary of DMAs, equitable distribution of water as per designed nodal demands with designed residual head can be achieved.	
22	**Transmission mains**	Design methodology for achieving economy in capital/pumping cost and equalisation of residual head at FSLs of ESRs is mentioned in detail in Chapter 6. By this method, velocities in pipes are increased to optimum level, diameters are reduced, pumping head is optimised and every ESR gets just designed quantity of water.	This methodology uses the tool of velocity (m/s) and head loss gradient (should not exceed 10m/km) prudently. The value of head loss gradient can be exceeded in hilly areas, however, the velocity should not exceed the permissible value of 2.5 m/s.
23	**Design of distribution system**	Design methodology in details is given in Chapters 12. Velocities in pipes need to be increased to optimum level and diameters can be reduced. Minimum and maximum velocity criteria are specified in section 6.6 in Part A of this manual.	Strategic points such as maximum and minimum ground elevation and the farthest point should be marked on the drawings of OZs/DMA.
24	**Bulk metering**	Bulk meters shall be installed at head work, inlet, and outlet of WTP and at entry of each DMA.	By observing minimum net night flow through bulk meter at inlet of DMA, Non-Revenue Water (NRW) can be effectively monitored.

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
25	**Automatic Meter Reading (AMR) meters**	It is recommended that bulk supply connection should have AMR meter installed for conducting water audit. Commercial establishment having connection size greater than 50mm and society of colony of high-rise buildings are encouraged to install AMR meters from the revenue generation perspective.	AMR facility is optional.
26	**Control valves** **PRVs** **FCVs**	PRVs are needed in hilly cities/areas. PRVs are also needed when some of the DMAs are situated on lower elevations. FCVs with dual Solenoid at entry of DMA are proposed. They should be set for peak hour design demand.	Control valves such as PRV and FCV are vital for equitable distribution of water and equal terminal pressures. FCV at entry of DMA helps in maintaining water level in the tank.
27	**Preparation of contract documents and speedy implementation**	Contract document for capital works need to be clear, unambiguously worded for avoiding litigation/arbitration/unrequired payment and speedy execution. This is achieved by formulating standardised (model) DTP and this avoids repetitive and erroneous work.	
28	**Break Pressure Tank (BPT)**	Design methodology of computing volume along with depth required is mentioned in section 6.14 in Part A of this manual.	Inlet and outlets should be kept at same elevation for BPT and MBR to

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
			optimise head on pumps and save electricity.
29	**Master Balancing Reservoir (MBR) & Zonal Balancing Reservoir (ZBR)**	The storage capacity of MBR for Urban area shall be designed for three hours of ultimate demand & for combined Urban & Rural as well as for Rural the storage capacity shall be three hours of ultimate demand. However, ULBs are free to carry out the capacity of MBR based on the mass curve. The storage capacity of zonal balancing reservoir in rural areas shall be designed for 2 hours capacity of the ultimate demand of the service tanks under its command area.	The capacity should be more than the downstream system volume (service tanks + pipelines) to run the system continuously.
30	**Sub-DMAs/Isolation valves**	For enabling effective break down maintenance of leaky pipes in distribution system, adequate number of isolation valves should be provided to isolate the network. Sub-DMA also helps to conduct water audit. Isolation valves should be such located that a segment of not exceeding 50 connections in hilly areas and 50 to 250 connections in other areas gets isolated for the purpose of repairs and rest of the connections remains unaffected. Optimisation of number of isolation valves is possible and	The drawing showing these locations of isolation valves should be readily available with maintenance staff. Modern softwares have facility of carrying out Criticality Analysis of the pipe network. Using this facility, optimum number of isolation valves can be determined. Formation of sub-DMAs with isolation valves are required in carrying out the STEP test.

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		recommended to operate the scheme on continuous supply basis.	
31	**Capacity of raw/clear water sump**	The capacity should be more than the downstream system volume (service tanks + pipelines) to run the system continuously. When WTP needs augmentation after 15 years, extra inlet from future Chlorine Contact Tank (CCT) to the clear water sump is required, which should be planned in the present WTP.	Two hours of the capacity of the WTP.
32	**Pipe material**	Distribution system – Provide metallic and/ or non-metallic pipes as per the site and service conditions. Raw/treated water pumping mains, transmission mains and feeder mains to DMAs - These are the arteries of water supply projects and preferably be laid with metallic pipe having internal lining. If non-metallic pipes are proposed, they shall be duly justified. Gravity transmission mains - Inside and outside city areas - pipes should be based on economical size of the gravity mains. The metallic pipes shall be preferred. If non-metallic pipes are proposed, they shall be duly justified.	

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
33	**Laying of pipelines**	Minimum cover of 0.9m is recommended, however cover should be provided as per respective BIS code for different pipe materials & suiting to the local field conditions Laying, jointing and alignment should be made as per the IS code. In the terrain where ambient temperature goes below 0 degree Celsius, pipes may be protected with proper insulation.	More than 25 mm size connection should be avoided to be given from small diameter such as 80 or 100 mm. Service connections must not be given from raw, pure water pumping mains, transmission mains, and mains feeding DMAs.
34	**Pipelines on both sides of roads having width 6 m and more**	In planning and design of new schemes, the roads having width 6 m or more, pipes are to be laid on either side of the road. This can also be done economically while deciding boundary of DMA.	It is necessary to lay pipelines on either side of the road so that while giving house connection, the road is not required to be cut/damaged. The method for roads having a width of more than 6 m is to insert the ducts intermittently in the body of the roads so that service connection pipes can be laid through it.
35	**Consumer underground tank**	For the buildings up to three floors, underground tank should not be encouraged at the customer's house.	This manual recommends considering 17-21 m residual head for Class I and Class II cities/towns and 12-15 m for other cities. For the buildings up to three storeys,

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		If such tank exists, then after stabilisation of 24×7 pressurised supply, such tanks shall be gradually removed/abandoned.	underground tank is not recommended at customer's house. If it is there, then after stabilisation of 24×7 pressurised supply, such tanks shall be removed/abandoned subsequently. However, for buildings with more than three storeys, they can have underground tank RCC/ PE with waterproof treatment to avoid outward seepage and inward contamination. The cleaning of such tanks is mandatory with frequency of once in six months and it should be strictly monitored by the agency responsible for O&M.
36	**Head loss computation**	Head loss can be computed using Hazen-Williams method or Darcy-Weisbach method.	
37	**Drinking water quality**	It shall be as per IS 10500:2012.	Drinking water criteria in Tables 1 to 6 from IS 10500:2012 are enclosed in **Annexure 2.5** of Part A Manual. The same is available along **with the latest amendments in Chapter 7** of Part A Manual.

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
38	**Express feeder for electric substations**	Express feeder for electric substations at pumping stations at headworks and at WTP as detailed in Chapter 16 at Sr no. 16.15 is mandatory to ensure continuous water supply in the city. The work of electric lines shall be got done from corresponding electricity board. Electricity Board shall not give electric connections to other customers from the express feeder. The cost of express feeder should be included in the project cost.	Express feeders from 11KV and above substation are necessary for uninterrupted electricity required for pumping water in 24×7 projects. The standby arrangement preferably from national power grid shall be provided. Standby in the form of generators may be provided for small BHP pumps up to 50 BHP.
39	**Consumer Survey**	Door-to-door consumer survey should be carried out. The consumer meters should be geo-tagged with GIS co-ordinates and shown on GIS maps of DMAs.	The city shall be divided into grid of suitable size. Survey team should visit all properties in an element of grid. During survey, illegal connections shall be identified.
40	**Physical Survey for generating Contours**	Ground elevations all along the roads in the city should be found out by total station method. The instrument should have capability of recording GIS co-ordinates. The elevation points shall be mapped in GIS and GIS-based contours shall be generated. If city terrain is not undulating, the contours can be generated using 3D stereo satellite method.	GIS based contours are necessary to assign the ground elevations to the nodes.

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
		In hilly areas when roads are not seen, “Drones” or other suitable methods may be used to generate contours.	
41	**Identifying Existing Pipelines and Condition Assessment**	Existing laid pipelines shall be identified by pipe alignment survey. Details are shown in Section 2.7.2 of this Chapter.	A change management team shall be formed comprising of ULB engineer, agency’s engineer, valve operators etc. They should identify existing pipes by interacting with local people.
42	**City Water Balance**	A city water balance considering IUWRM may be computed.	Refer Section 4.14 of Part A Manual
43	**Design of buried pipelines in seismic active areas**	The design shall be as per provisions of “IITK-GSDMA Guidelines For Seismic Design of Buried Pipelines Provisions with Commentary and Explanatory Examples”, which is available at http://www.iitk.ac.in/nicee/IITK-GSDMA/EQ28.pdf In seismic prone areas, MS pipes may be used for water supply projects as mild steel is flexible. DI pipes, being semi-rigid, can also be used with restraint joints.	The seismic hazards which are directly related to pipeline failure can be classified as: Permanent ground deformation related to soil failures Longitudinal permanent ground deformation Transverse permanent ground deformation Landslide Buoyancy due to liquefaction

S. No.	Parameter	Conversion from Present Intermittent Supply to 24×7 Pressurised Water Supply System	Remarks
1	2	3	4
			Permanent ground deformation related to faulting Seismic wave propagation
44	**Branch roads to WTP, Head works, MBR, BPT, ZBR**	All pipelines should be laid along all season roads; missing links and branch roads should be provided to important structures at project cost.	Pipelines should not be laid along cross country for saving lengths.

Table 2.8: Recommended norms for O&M works

S. No.	Parameter	Conversion from present intermittent supply to 24×7 pressurised water supply system	Remarks
1	2	3	4
1	**NRW monitoring and control measures (leakage programme)**	Since bulk meters at the entry of DMAs and 100% consumer meters are to be installed, and active leakage management programme is essential, the NRW values can be computed by (a) knowing the quantity of water entering DMA and consumption in DMA); (b) conducting step tests; (c) NRW of the entire system should be brought down to 15% or less; (d) NRW monitoring measure using water meter and communication technology are provided in Chapter 14 of Part A of this manual.	In the passive leakage programme, only visible leaks are attended and repaired. For leakage identification, modern methods such as detection using inert gas techniques can be used, which can be conducted in a shorter time compared to the conventional methods.
2	**Creation of NRW cell**	Mandatory for all the cities and towns along with quick response teams with vehicles equipped with necessary tools/equipment.	Dedicated NRW cell is required which can take stock of situation and continuously monitor NRW levels.
3	**Creation of calibration/repair workshop for domestic consumer meters**	ULB should promote the creation of a calibration/repair workshop for domestic consumer meters for 15 mm to 50 mm diameters with bench testing facility on the lines of the electricity board. Adequate stock of common spare parts should be ensured for making them commercially viable.	ULB should promote the creation of a meter repair workshop with a testing facility.
4	**Water audit**	Due to the provision of bulk meter at the entry of DMA, NRW of the OZ can be computed as all consumer connections are equipped with meters. Water audit of rising mains, transmission mains, OZ, and DMAs is essential.	In a 24×7 system, a water audit is a continuous activity. There is an 'economic level' of reducing NRW to 10% in the distribution systems at DMA level.
5	**Energy audit**	Energy audit is essential as per IS 17482:2020.	In many ULBs, pumps are not replaced even after 15 years.

S. No.	Parameter	Conversion from present intermittent supply to 24×7 pressurised water supply system	Remarks
1	2	3	4
			Hence, low efficiency is observed, and ULB has to pay more electricity bills.
6	**Eradication of illegal connections**	It is certainly possible to eliminate all illegal connections by enlisting suspected connections in a house-to-house survey to be undertaken. Step by step, illegal connections can be eliminated.	Identification of illegal connections should be made during customer surveys and mapped on GIS.
7	**Water quality**	Water quality should be monitored as per IS 17482:2020.	Water quality testing facilities should be created.
8	**SCADA**	SCADA system is recommended for cities (preferably population more than 10 Lakhs) to monitor the flow and functioning of the water supply systems, including night flow and leakages.	All the level controls of tanks, pumps, Bulk meters, FCVs, and PRVs should be connected to the SCADA. Softwares compatible to SCADA may be used to monitor real-time values of concentration of residual chlorine in any pipe at any point of time.
9	**Digital twin**	Digital twin technology may be adopted which uses real-time data generated by SCADA. With data analytics, digital twin makes predictive analysis. Thus, digital twin can help ULB to mitigate any urgencies such as pump failure, pipe burst, fire outbreak, low pressures, or the failure of ageing assets.	"Digital twin" is a virtual representation of ULB's water supply system. Digital twin brings SCADA, GIS, hydraulic modelling, and consumer information into a connected data environment, delivering cost-effective operations strategies in real time.
10	**Consumer billing and complaint redressal**	Consumer billing and complaint redressal system is essential. Computerised billing systems should be encouraged.	With SCADA/MIS, it is possible to show the redressal of complaints online for compliance of complaints. Complaint redressal cell should be set up.

S. No.	Parameter	Conversion from present intermittent supply to 24×7 pressurised water supply system	Remarks
1	2	3	4
11	**Special Purpose Vehicle (SPV)**	SPV may be preferred by the city to implement 24x7 water supply project alongwith long-term O&M.	Details are given in Part C of the Manual.
12	**PPP/O&M Through Contractor**	AMRUT 2.0 recommends planning and implementation of projects in PPP mode in water sector in cities with population more than 10 lakhs. It is recommended to develop standardised tender documents for various sub-works of O&M of headworks, pipelines, WTP and pumping machinery, etc.	Some of the components like WTP, pumping machinery with transformer, major pipeline, distribution system, etc., may be undertaken using separate O&M contracts.
13	**Training and Capacity Building**	Various training modules as discussed in the advisory on “GIS Mapping of Water Supply and Sewerage Infrastructure", as well as the PHE training program conducted by CPHEEO, may be referred to.	

CHAPTER 3: PROJECT REPORTS

3.1 Introduction

All the projects go through various stages between the conceptualisation till the time of completion and commissioning of the project. The important stages are as follows:

(i) Identification of a project - for some projects where existing system is available, pre-feasibility can be carried out as a part of the feasibility report (refer Section no. 3.6)

(ii) Execution of a Pre-feasibility study;

(iii) Preparation of a feasibility report (population projections, source availability, conditional assessment of the infrastructure, land availability for all component sites, concept development, alternatives, technological options, funding, revenue generation, operation and maintenance (O&M) expenditure, asset management etc.)

(iv) Preparation of a Detailed Project Report (DPR) including GIS survey, collection of data, GIS mapping of existing infrastructure, consumer survey including geo coding of consumer meters, raw water quality characteristics. The conditional assessment of existing infrastructure, population projections of city, ward-wise population forecast, supply, demand forecast, demand allocation to nodes of distribution network using GIS based land use patterns, capacity/sizing of various components, viz., WTP, ESR/GSR, network, etc., should be included. Special emphasis to be given to source identification, source sustainability, selection of treatment technology/ method, land availability for all components of the system, electrical feeder availability, environmental social safeguards. The detailed engineering design including layouts, hydraulic flow diagram, single line diagrams, GIS-based network modelling incorporating zoning and District Metering Areas (DMAs), rehabilitation plan of existing infrastructure, system improvement plan, estimation and costing, O&M plan, financial analysis, and revenue generation, etc.

(v) Technical appraisal and financial and administrative sanctions or approvals, including various permissions needed from concern departments, viz., water resources, highways, railways, forest, etc.

(vi) Execution/Implementation of the project (bidding, contract award and project management)

(vii) O&M

3.2 Project Reports

Project reports deal with all the aspects of pre-feasibility planning and establishes the need as well as the feasibility of projects technically, financially, socially, culturally, environmentally, legally, and institutionally. Project report should be prepared in four stages, viz., (i) identification stage; (ii) pre-feasibility stage; (iii) feasibility stage; and (iv) DPR stage. Detailed engineering and preparation of technical specifications and tender documents are not necessary for taking investment decisions since these activities can be carried out once source and financial sustainability is ensured. At the end of each stage, decision on broad technical and financial feasibility should be taken into consideration while deciding whether to proceed to the next planning stage and commit the necessary manpower and financial resources for the next stage. The basic design of a project is influenced by the authorities/organisations who are involved in approving, implementing, and operating and maintaining the project. Therefore, the institutional arrangements through which a project will be brought into operation must be decided at the project preparation stage itself. Sometimes more than one organisation may have a role to play in the various stages of preparation of a project, it is therefore, necessary to identify a single entity to be responsible for overall management and co-ordination of each stage of project preparation. The implementing authority and authority responsible for O&M of a project should be consulted at the project preparation stage itself.

3.3 Project Identification Report

The identification of the project is based on the existing infrastructure and need of additional infrastructure to attain Service Level Benchmarks (SLBs, as published by MoHUA). The project identification report provides an overview of the existing water supply systems, the need for the project, and a brief description of the indicated project and its alternatives and order-of-magnitude costs. At this stage, the planner explains the project and its priority within the context of ULB, state, regional and national development plans for the sector.

The project identification report can be prepared in a reasonably shorter time, if the planner is familiar with the local, sectoral, and regional development plan, and sector programme is available. Where there is considerable information already available and some analysis has already been carried out, such a knowledgeable planner should be able to produce the report based on a "desktop study". It is essential, however, that the project area and the site is inspected to ensure that existing background information is realistic including confirmed sustainable source, land availability and that future developments are unlikely to provide any surprises/challenges to project planners. If there is little existing data and analysis, some block estimates of necessary facilities and land acquisition/resettlement cost (if any) will have to be made. If new technologies are being considered for treatment. ULBs can first go for pilot studies. The following checklist shows the kind of information which should be included in a Project Identification Report:

(i) Identification of the project area and its physical environment;

(ii) Provision a GIS map showing the project area, project components, and a definition of the intended beneficiaries. The following plans may be enclosed with the report:

    a. an index plan to a required scale of 1 cm = 2 km or so, showing the project area, existing works, proposed works, location of community/township or institutions to be served;

    b. a schematic diagram showing the salient levels of project components;

(iii) Analysis of the existing population, its physical distribution and socio-economic factors;

(iv) Identification of the present water supply arrangements and status of SLBs in the project area including the baseline performance indicators, gap between the benchmark and the actual performance indicator, population projections including ward-wise population projection, for planning period according to existing and future land use plans or master plans;

(v) Evaluation of water availability and requirements during project horizon for domestic, industrial, commercial, institutional and any other uses;

(vi) Establishment of the need of the project in respect of local, regional, national context. State the objective of short-term and long-term plans in terms of population to be served, SLB to be achieved and the impact of the project after implementation;

(vii) Alignment of sectoral strategies with ongoing related activities;

(viii) Identification of any adverse impacts on the environment and positive impact on the livelihood of the proposed beneficiaries of project area;

(ix) Examination of the master plan for present and future requirement of infrastructure for various project components, with alternatives for physical facilities and supporting activities (O&M, capacity building, etc.);

(x) Presentation of preliminary cost estimates (component-wise) for pre-construction activities (e.g., project preparation cost, land acquisition/resettlement cost, etc.), construction of physical facilities, supporting activities and cost of O&M, consumer services, etc. Also, identify the source of funding for financing capital works and work out plan for probable financial burden on the ULB as per annual revenue and expenditure calculated;

(xi) Indication of institutions responsible for project preparation, project approval, financing, implementation, O&M, viz., ULB, State Government, and National Government;

(xii) Outline water-related policy issues that need to be addressed prior to the project approval;

(xiii) Indication of challenges with respect to technical capacity of the implementing agency required for next stage that may become an obstacle;

(xiv) Specification of the preliminary terms of reference for the pre-feasibility and feasibility stages of the project preparation.

3.4 Survey and Investigations

Once the project is approved in-principle based on the Project Identification Report, the survey and investigation must be carried out in full details, to plan and design the components of proposed water supply system.

The details of all the survey and investigation are referred in Section 2.7 of Part A of this manual and covering the following:

- Basic information
- Physical aspects
- Survey of natural conditions
- Sanitary survey of sources
- Asset surveys and condition assessment of existing facilities
- Detailed project survey including population, water demand, land availability, asset availability from existing water supply scheme for new project, pipeline network, identification of source and its sustainability for future demand
- Digital terrain modelling
- GIS mapping
- Geotechnical investigations

3.5 Environmental and Social Safeguards studies

The development of water supply projects and programmes has a wide range of environmental and social impacts, both beneficial and adverse. The safeguard measures are designed to first identify and then try to avoid, mitigate, and minimise adverse environmental and social impacts that may arise in the implementation of development projects. The studies have to be carried out to avoid delays in the execution and implementation of the project.

3.5.1 Environmental Safeguards

Environmental safeguards aim to ensure the environmental soundness and sustainability of projects, and to support the integration of environmental considerations into the project decision-making process. The project impact and their significance have to be identified, alternatives have to be examined, and environmental impact management plans have to be prepared, implemented, and monitored. The people likely to be affected by the project are also consulted. The costs involved in environmental safeguards can be arrived at and included in the project cost.

3.5.2 Social Safeguards

Major development projects frequently have adverse implications that harm vulnerable communities. Projects that are likely to evict families from their homes, deteriorate Indigenous peoples' living conditions, or aggravate social problems on a local level.

Social safeguards help development programmes avoid negative consequences, manage social risks, and encourage social inclusion.

Social safeguards are meant to prevent these and other unforeseen consequences, and to devise appropriate strategies to minimise them when they cannot be avoided. They also enable projects develop their full potential, manage social risks, and promote social inclusion. The costs involved in social safeguards can be arrived at and included in the project cost.

3.6 Pre-Feasibility Report

After technical and administrative clearance is accorded to the project identification report by the concerned authority and/or owner of the project, and commitments are made to finance further studies, the work of preparation of pre-feasibility report should be undertaken by an appropriate agency. The agency may be State/UT Urban Development Department or Water Supply Department/Board/Urban Local Body, or other similar agencies. Professional consultants working in the water supply sectors may also be engaged by the Agency. The terms of reference and the scope of the project preparation should be carefully set out.

Since feasibility studies are time extensive and expensive, the essence of the pre-feasibility stage is the screening and ranking of all project alternatives to select the preferred project before the detailed feasibility evaluation continues. This logic should be followed whether the pre-feasibility report is a separate activity, is an interim report towards a full feasibility study, or is included with the findings of the feasibility stage in a single report. The pre-feasibility study may be a separate and discrete stage of project preparation, or it may be the first stage in a comprehensive feasibility study. A pre-feasibility report can be taken to be a Preliminary Project Report, the structure and component of which are as follows:

(i) Executive summary

(ii) Introduction

(iii) The project area, its selection, and the need for a project

(iv) Proposed “Drink from Tap” with 24×7 water supply systems project

(v) Financial, environmental, and social analysis

(vi) Conclusions and recommendations

(vii) Tables, figures/maps, and annexures

3.6.1 Executive Summary

It is a good practice to provide an executive summary at the beginning of the report. The executive summary provides a brief overview of the project and contains its main points, salient features, basic strategy, and approach adopted in developing the study project. It is a summarised version of a complete project.

The objective of achieving "Drink from Tap" with 24×7 pressurised water supply system has to be clearly mentioned with ULB's intentions and proposed actions planned to be taken.

3.6.2 Introduction

This section briefly explains the origin and concept of the project, how it was prepared and the scope and status of the report. The sub-sections may be detailed as under:

a) Project Genesis:

(i) Describe how the proposed project idea was developed and its alignment with current related policies of development.
(ii) Indicate the agency responsible for promoting the project and their roles.
(iii) List and explain previous studies and reports on the project (particularly the project identification report) prepared by different agencies.
(iv) Refer to related long-term plans for the sector, regional development, land use, water resources sustainability, environmental and social safeguards, public health, etc.
(v) Explain the Methodology adopted for carrying out the study.
(vi) Outline the study's timelines.

b) Scope and intended use of the Report:

(i) Explain how this pre-feasibility report fits in the overall process of project preparation.
(ii) Identify data limitations.
(iii) List interim reports or notes submitted during the pre-feasibility study and summarise any guidance provided by the responsible project authority.
(iv) Explain whether the pre-feasibility report is intended to be used to obtain in-principle approval for the proposed project. If so, the report needs to be more comprehensive and less tentative in its conclusions than in cases where a feasibility study is already underway or expected to be initiated shortly after the pre-feasibility report is completed.

3.6.3 The Project Area and the Need for the Project

This section explains why the project is needed and talks about the following:

(i) the project area and population served;

(ii) the present water supply services in the project area;

(iii) the prospects for future development;

(iv) the need to improve existing services.

3.6.3.1 Project area

- Give a geographical description of the project area with map/maps, describe special features such as topography, climate, culture, religion, migration, etc., which may affect project design, implementation, O&M.

- Provide GIS Map showing administrative and political jurisdiction;
- Include details of notification of additional towns/villages as urban area, if any;
- Describe, if any, ethnic, cultural, or religious aspects of the communities that may have a bearing on the project proposal.
- Show coverage areas where the pipe network is expected and mark the areas where a pipeline is not expected to be laid (for example cantonment area, industrial area, etc.).

3.6.3.2 Population pattern

- Estimate population in the project area, indicating the source of data or the basis for the estimate.
- Review previous population data of the project area, historic growth rates and its causes.
- Estimate future population growth with different population forecasting methods and indicate the most probable growth rates and compare with past population growth trends.
- Adopt computation of ward-wise future population density based on equivalent area (GIS based application) may for projection of city population. Population projected using various methods should be analysed and considered judiciously.
- Estimate probable densities of population in different parts of the project area at future intervals of time, e.g., five, ten, fifteen, twenty, and thirty years ahead.
- Compare growth trends within the project area, with those for the region, state, and the entire country.
- Discuss other factors likely to affect the population growth rates in the project area such as development marked for the area in the master/regional plans that may increase or decrease the growth rate, e.g., national park, special economic zone (SEZ), industrial parks, industrial corridors, proposed merger of adjoining villages, etc.
- Discuss patterns of seasonal migration, if any, and estimate floating population within the area. Indicate implication of the estimated growth pattern on housing and other local infrastructure.

3.6.3.3 Economic and social conditions

- Describe present living conditions of the people of different socio-economic and ethnic groups and their likely uplift in the future.
- Identify locations according to income levels or other indications of socio-economic studies.
- Show on the project area map ward-wise density of population, and the present and future land uses (as per the development plan).
- Provide information on housing conditions and relative proportions of owners and tenants.
- Provide data on education, literacy, and unemployment by age and gender.
- Provide data and project housing standards, and average household occupancy in various parts of the project area.
- Describe public health status within the project area, with particular attention to diseases related to water and sanitary conditions; provide data on maternal and infant-mortality rates, and life expectancy.
- Provide status of health care programmes in the area, as well as other projects, which have bearing on improvements in environmental sanitation.

3.6.3.4 Institutions involved

- Identify the institutions (government, semi-government, non-government, etc.) which are involved in any of the stages of water supply systems project development in the area (planning, preparing projects, financing, implementation, O&M, and evaluation).
- Comment on roles, responsibilities, and limitations (territorial or others) of all the identified institutions, in relation to water supply systems (this may also be indicated on a diagram).
- Outline various institutions involved in granting permissions for implementation of the water supply projects for, e.g., Water Resources Department/Ground Water Development Authority for water source availability, forest department for pipeline alignment through forest area, national/state highways departments for alignments along or across highways, railway crossings, etc. The process and costs involved for availing the permission/s has to be clearly mentioned.

3.6.3.5 Available water resources

- Summarise the quantity and quality of surface and ground water resources, actual and potential, in the project area and vicinity (give information of sources).
- Indicate studies carried out or being carried out concerning development of potential sources, and their findings.
- Describe the existing patterns of water use by all sectors (irrigation, industrial energy, domestic, etc.), and comment on supply surplus or deficiency and possible conflicts over the use of water, at present and in future.
- Discuss any pollution problems, if any, which might affect available surface and ground water resources.
- Assess sustainability of water resources and propose suggestive measures to ensure sustainability.
- Mention the role of agencies/authorities responsible for managing water resources, allocation, and quality control.

3.6.3.6 Existing water supply systems and population served

Describe all the existing water supply systems in the project area, indicating the details as under:

- source of water, quantity and quality available in various seasons, components of the system such as head works, transmission mains, pumping stations, treatment works, balancing/service reservoirs, distribution system, reliability of supply in all seasons;
- areas supplied, hours of supply, water pressures, operating problems, bulk meters, metered supplies, un-metered supplies, bulk supply connections, AMR connections, supply for commercial use, industrial use, and domestic use;
- additional sources for water supply such as, wells, tube wells, bores, water vendors, other authorities, e.g., state industrial development corporations, etc.;
- information of number of Operational Zones (OZs) and DMAs in each OZ;
- number of people served according to water supply systems of the following category:
  - unprotected sources like shallow wells, rivers, lakes, ponds, etc.;
  - protected other sources like wells, tube wells, bores, rainwater storage tanks etc.;
  - areas not served by distribution network.
- number of household tap connections, number of stand-posts and percentage of population served with household tap connection and stand-post, if any;
- consumers' opinion about stand-post water supply, (e.g., distance, hours of supply, waiting time etc.) and their aspiration for household tap connection;

- number of people obtain water from more than one source, note these sources, and their water used, e.g., drinking, bathing, washing, etc., and reasons for their preferences;
- explain non-revenue water (NRW), probable causes and trends and efforts made to reduce NRW;
- engineering and social problems of existing systems and possible measures to resolve these problems and the expected improvement of the systems.

3.6.3.7 Existing sanitation systems and population served

Even if the proposed project may be for providing a single service, i.e., water supply and not sanitation, the existing sanitation arrangements should be described, giving details of the existing sanitation and waste disposal systems in the project area, and the number of people served by each system. Impact of existing system on drinking water quality and environment should be assessed and details provided for contamination events occurred.

Briefly describe existing systems of storm water drainage and solid waste collection, treatment, and disposal. This discussion should be focused in terms of their impact on water supply systems and environment.

3.6.3.8 Need for the project

The following may be included:

- Describe as to why the existing system cannot satisfy the existing and projected demands at the desired SLBs to the population, commercial, institutional, and industrial demand with adequate quantity and quality on long term basis.
- Describe the consequences of not taking up a project for rehabilitation/ augmentation of the existing system and/or developing a new system.
- Indicate priorities for improvement of existing system, expansion of system, construction of new system, supply for domestic, industrial, and commercial and institutional use.
- Assess the need for consumer education in hygiene.
- Comment on the urgency of project preparation and implementation.

3.6.4 Long Term Plan for Water Supply

(i) **Water supply services improvement**

Improvement in water supply services has to be planned as a phased development programme keeping in view of consistency with the future overall development plans associated with term project or strategic plan. The implementation should be made as an integrated programme for all components of the water supply systems. A long-term plan may be prepared for a period of 30 years, and alternative development sequences may be identified to provide target service coverage and standards at affordable costs. From these alternative development sequences, a priority project to be implemented in near-term can be selected. It is this priority project, which then becomes the subject of a comprehensive feasibility study.

(ii) **Service Coverage**

The planning of new water supply schemes shall be made for “Drink from Tap” 24x7 pressurised water supply system basis to achieve the SLBs for water supply systems, released by the Ministry of Housing and Urban Affairs, Govt. of India, from time to time. Redevelopment or retrofitting of existing water supply infrastructure should also be adopted to achieve the SLBs.

Alternative development sequences should be identified in the light of the service coverages to be achieved during the planning period in phases. This calls for definition of the following:

- population to be covered with improved water supply facility with adequate quantity and of prescribed quality on long term basis;
- other consumers of water to be covered (industrial, commercial, government, institutions, etc.);
- service standards to be provided for various section of population, e.g., functional household tap connections (FHTC), yard-taps, bulk connections, public kiosk, utility services and temporary point sources, etc.;
- target dates by which the above-mentioned service coverage would be extended within the planning period, in suitable phases.

(iii) **Project affordability**

It must be noted that service standards can be upgraded over a period of time. Therefore, various options can be considered for different areas. While selecting a service standard, community preferences and affordability should be ascertained through a dialogue with the intended beneficiaries. Only those projects, which are affordable to the people they serve must be selected. This calls for careful analysis of the existing tariff policies and practices, cost to the users for various service standards, willingness to pay and income of various groups of people in the project area.

(iv) **Water requirement**

Achieve the service coverage in stages over a planned period, requirements of water can be worked out for each year (or in suitable stages), by adopting different standards at different stages. The demand for industrial, commercial, and institutional users may also be added. Thus, water for the projected needs throughout the planned period can be quantified, (duly considering realistic allowances for unaccounted for water and the daily and seasonal variations) for alternative service standards, and service coverage. These demands form the basis for planning and providing system requirements.

The annual water requirements should also take into consideration water demands for upgrading sanitation facilities if proposals to that effect are under consideration. Consistency and co-ordination have to be maintained between projections for both water supply and sanitation services.

(v) **Anticipation of funds**

It must be noted that availability of funds, through various missions of the central government, states/UTs government/loan or grant from bilateral and multilateral agencies, private investment, public-private partnership, or any other sources, is one of the prime factors that will ultimately decide the scope and scale of a feasible project.

(vi) **Selection of a strategic plan:**

Each of the alternative development sequences, which can overcome the existing deficiencies and meet the present and future needs, consists of a series of improvements and expansions to be implemented over the planned period. Since all needs cannot be satisfied in immediate future, it is necessary to carefully determine priorities of target groups for improvement in services and stages of development and thus restrict the number of alternatives.

(vii) **Planning for system requirement**

The following needs to be considered as part of planning:

- Possibilities of rehabilitating and/or de-bottlenecking the existing systems
- Reduction in water losses which can be justified economically, by deferring development of new sources
- Alternative water sources, surface and ground water with particular emphasis on maximising the use of all existing water sources
- Alternative transmission and treatment systems and pumping schemes
- Distribution system including pumping station, balancing/service reservoirs and adoption to “Drink from Tap” with 24×7 pressurised water supply systems, with DMA approach. The details can be referred from Section 2.8 of Part A manual.
- Providing alternative service standards in future, including upgrading of existing facilities and system expansion

(viii) **Need Assessment for Supporting Activities**

It may also be necessary to ascertain if supporting activities like Information, Education and Communication (IEC), health education, staff training and institutional improvements, etc., are necessary to be included as essential components of the project. All the physical and supporting inputs need to be carefully costed (capital and operating), after preparing preliminary designs of all facilities identified for each of the alternative development sequences. These alternatives may then be evaluated for the least cost solution by net present value method, which involves:

- expressing all costs (capital and operating) for each year in economic term;
- discounting future costs to present value;
- selecting the sequence with the lowest present value by net present value method.

(ix) **Costings and their expressions**

As stated above, costs are to be expressed in economic terms and not in terms of their financial costs. This is because the various alternatives should reflect resource cost to the economy as a whole at different future dates. Costing of the selected project may, however, be done in terms of financial costs, duly considering inflation during project implementation.

3.6.5 Proposed Water Supply Project

(i) Details of the Project

The project to be selected are those components of the least cost alternative by net present value method of development sequence, which can be implemented during the next two to four years. Components of the selected project may be as follows:

- Rehabilitation, retro-fitting and de-bottlenecking of the existing facilities for providing “Drink from Tap” with continuous (24×7) water supply systems
- Construction of new facilities for improvement and expansion of existing systems
- Support activities like information, IEC, consumer education, public motivation, etc.
- Equipment and other measures necessary for O&M of the existing and expanded systems

- Consultancy services needed (if any) for conducting feasibility study, detailed engineering, construction supervision, socio-economic studies, environment and social safeguards studies, studies for reducing water losses (NRW reduction), tariff studies, willingness to pay, acceptance of metering, studies for improving accounts support activities

(ii) Support documents

All project components should be thoroughly described, duly supported by documents such as:

- GIS-based project area map with clear demarcation of ward boundary;
- technical information for each physical component (infrastructure), socio-economic study, statutory clearances and economic analysis, where necessary;
- preliminary engineering designs (hydraulic design) and drawings in respect of each physical component, such as head works, transmission mains, pumping stations, treatment plants, balancing reservoirs, distribution lines, etc.

(iii) Implementation schedule

A realistic implementation schedule should be presented, taking into consideration time required for all further steps to be taken, such as conducting feasibility study, appraisal of the project, sanction to the project, fund mobilisation, various permissions needed, implementation, trial runs, and commissioning. In preparing this schedule, due consideration should be given to all authorities/groups whose inputs and decisions can affect the project and its timing, bottlenecks expected during execution of the project, time required for getting statutory approvals, No Objections Certificates (NOCs), and other necessary components.

(iv) Cost estimates

Cost estimates of each component of the project should be prepared and annual requirement of funds for each year should be worked out, taking into consideration the likely annual progress of each component. Due allowance should be made for physical contingencies and annual inflation. This exercise will result in arriving at total funds required annually for implementation of the project.

(v) Environment and social impact

The pre-feasibility report should bring out any major environment and social impact the project is likely to cause and if these aspects will affect its feasibility.

(vi) Institutional responsibilities

The pre-feasibility report should identify the various organisations/departments/ agencies who would be responsible for further project planning, preparation, approval, sanction, funding, implementation, O&M of the project. This should also indicate the strength of personnel needed to implement and later operate and maintain the project. It should also discuss special problems likely to be encountered during O&M, in respect of availability of skilled and technical staff, training and professional development required, funds, transport, consumables, communication, power, spare parts, etc. Quantitative estimates of all these resources should be made and included in the project report.

(vii) Financial aspects

The capital cost of a project is a sum of all expenditure required to be incurred to complete design and detailed engineering of the project, construction of all its components, including support activities and conducting special studies. After estimating component-wise costs, they may also be worked out on annual basis, throughout the implementation period, taking into consideration construction schedule and allowances for physical contingencies and inflation. Basic item costs to be adopted should be of the current year. Total of such escalated annual costs determines the final cost estimate of the project. Financing plan for the project should then be prepared, identifying all the sources from which funds can be obtained, until the project is completed. The possible sources of funds include:

- cash reserves available with the project authority;
- cash generated by the project authority from sale of water from the existing facilities;
- grant-in-aid from the Government;
- loans from the Government;
- loans from Indian financing institutions, banks, etc.;
- loans and grants-in-aid from bilateral and multilateral funding agencies like AFD, World Bank, JICA, ADB, etc.;
- open-market borrowings, e.g., bonds;
- public-private partnership (PPP);
- capital contributions from company social responsibility (CSR), voluntary organisations, etc.

If the lending authority agrees, interest payable during implementation period can be capitalised and loan amount increased accordingly.

The next step is to prepare recurring annual costs (annual operating budget) of the project for the next few years (say five years) covering the operating and maintenance expenditure of the entire system (existing and proposed). This would include expenditure on staff, chemicals/consumables, energy, spare parts and other materials for system operation, transportation, up-keep of the systems and administration.

The annual financial burden imposed by a project comprises the annual recurring cost and payment towards loan and interest (debt servicing). This has to be met from the operational revenue, which can be realised from sale of water. The present and future tariff of water should be identified and a statement showing annual revenue for five-year period, beginning with the year when the project will be operational, should be prepared. If this statement indicates that the project authority can generate enough revenue to meet all the operational expenditure as well as repayment of loan and interest, the lending institution can be persuaded to sanction loans for the project.

Every state government and the Government of India have programmes/missions for financing water supply schemes in the urban and rural areas, and definite allocations are normally made for the national plan periods. It will be necessary at this stage to ascertain if and how much finance can be made available for the project under consideration, and to estimate annual availability of funds for the project till its completion. This exercise has to be done in consultation with the concerned department of the Government and the lending institutions, who would see whether the project fits in the sector policies and strategies, and can be brought in an annual planning and budgetary cycle taking into consideration the commitments already made in the sector and the overall financial resources position. The project may be finally sanctioned for implementation if the financing plan is firmed up.

3.6.6 Conclusions and Recommendations

(i) Conclusions

This section should present the essential findings and results of the pre-feasibility report. It should include a summary of:

- Review of the need for the project;
- Existing service coverage and SLBs;
- Long-term development plans considered;
- The recommended project, its scope in terms of service coverage and SLBs;
- Priorities concerning target-groups and areas to be served by the project;
- Capital costs and tentative financing plan;
- Annual recurring costs and debt servicing;
- Tariffs and projection of operating revenue;
- Limitation of the data/information used, and assumptions and judgments made; need for in-depth investigation, survey, and revalidation of assumption and judgments, while carrying out feasibility study.

The administrative difficulties likely to be met with and risks involved during implementation of the project should also be commented upon. These may pertain to boundary question for the project area, availability of water, sharing of water sources with other users, availability of land for constructing project facilities, permissions from various agencies, co-ordination with the various agencies, acceptance of service standards by the beneficiaries, acceptance of recommended future tariff, shortage of construction materials, implementation of support activities involving peoples' participation, supply of power, timely availability of funds for implementation of the project, and problems of O&M of the facilities.

(ii) Recommendations

a. This should include all actions required to be taken to complete project preparation and implementation, identifying the agencies responsible for taking these actions. A detailed timetable for actions to be taken should be presented if found necessary and feasible, taking up of works for rehabilitating and/or de-bottlenecking the existing system should be recommended as an immediate action. Such works may be identified and costed so that detailed proposals can be developed for implementation.
b. The proposal of project authority for taking up detailed investigations, data collection and operational studies, pending undertaking, and feasibility study may also be indicated.
c. The feasibility study can then be taken up at the beginning of the implementation phase and results of the study, if noticed to be at variance with the earlier ones, suitable modification may be introduced during implementation.
d. With respect to projects, a comprehensive feasibility study may have to be taken up before an investment decision can be taken.

3.7 Feasibility Report

The feasibility report may have the following sections:

(i) Background

(ii) The proposed project

(iii) Institutional and financial aspects

(iv) Conclusion and recommendations

3.7.1 Background

This section should describe the history of project preparation, the relation of this project to studies carried out earlier and, in particular, set in the context of a pre-feasibility report. It should also bring out if the data/information and assumption made in the pre-feasibility report are valid, and if not, changes in this respect should be highlighted. References to all previous reports and studies should be made.

In respect of the project area, the need for a project and strategic plan for water supply, only a summary of the information covered in the pre-feasibility report should be presented, highlighting such additional data/information collected, if any, for this report. The summary information should include the planning period, project objectives, service coverage, SLBs considered and selected for long-term planning and the project, community preferences, and affordability, quantification of future demands for services, alternative strategic plans, their screening and ranking, recommended strategic plan, and cost of its implementation.

3.7.2 The Proposed Project

This section describes details of the project recommended for implementation. The information presented here is based on extensive analysis and preliminary engineering designs of all components of the project. The detailing of this section may be done in the following sub-sections:

(i) Objectives

Project objectives may be described in terms to achieve the objectives such as “Drink from Tap” with 24×7 pressurised water supply system, SLBs, functional household tap connection, health status improvements, ease in getting water by consumers, improved living standards, capacity building, institutional improvements, etc.

(ii) Project users

Define number of people by location and institutions/industrial units who will benefit from the project area and reasons for the same, and explain user’s involvement/participation during preparation, implementation, and O&M of the project.

(iii) Rehabilitation and de-bottlenecking of the existing water supply systems

In fact, rehabilitation, improvements, and de-bottlenecking works, if necessary, should be planned for execution before that of the proposed project. If so, these activities should be mentioned in the feasibility report. If, however, these works are proposed as components of the proposed project, the necessity of undertaking the rehabilitation/improvement/de-bottlenecking works should be explained.

(iv) Project description

This may cover the following items:

- Definition of the project in the context of the recommended development alternative (strategic plan) and explanation for the priority of the project;
- Details of existing infrastructure which shall be put in service;
- Brief description of each component of the project, with maps and drawings;
- Brief description of measures to be taken to achieve “Drink from Tap” with 24×7 water supply, SLBs, including the functions, location, design criteria, and capacity of each component;
- Technical specification (dimension, material) and performance specifications;
- Stage of preparation of designs and drawings of each component;

- Method of financing and constructing in-house facilities, like plumbing and service connection, etc.

(v) Support activities

Need for description of components such as IEC, capacity building, and other stakeholders training; water quality testing and surveillance; improving billing and accounting; public awareness, consumer services, health education; community involvement/participation, etc.; and timing of undertaking these components and the agencies involved.

(vi) Integration of the proposed project with the existing and future systems

Describe how the various components of the proposed project would be integrated with the existing and future works to achieve the objects and purpose of the project.

(vii) Agencies involved in project implementation and relevant aspects

- Designate the lead agency (Implementing Agency).
- Identify other support agencies including government agencies who would be involved in project preparation and implementation, describing their roles, such as granting administrative approval, technical sanction, permissions, approval to annual budget provision, sanction of loans/grants and other funding agencies, and convergence of funds, construction of facilities, procurement of materials and equipment, etc.
- Outline of arrangements to co-ordinate the working of all concerned agencies with special attention needs to be on co-ordination with the road, railway, electricity, telecommunication, forest, and municipal authorities to get necessary permissions on time to avoid delay in implementation.
- Designate the operating agency and its role during the implementation stage;
- Define the role of Project Management Consultants (PMCs), if necessary, including the scope of their work and terms of reference;
- Describe regulations and procedures for procuring key materials and equipment, power, and transport problems, if any.
- Estimate the number and type of workers and their availability;
- Specify procedures for fixing agencies for works and supplies and the normal time it takes to award contracts.
- List any imported materials, if required, and outline a procedure to be followed for importing them, including an estimation of the delivery period, if any;
- Outline any legislative and administrative approvals required to implement the project, such as those pertaining to riparian rights, allocation of water reservation and point of allocation, water quality criteria, acquisition of lands, permission to construct across or along roads and railways, high-tension power lines, in forest area and defence or other such restricted areas.
- Offer comments on the capabilities of contractors and quality of material and equipment available indigenously.

(viii) Cost Estimates

- Outline basic assumptions made for unit prices, physical contingencies, price contingencies, and escalation.
- Create a summary of the estimated cost of each component for each year till its completion and work out total annual costs, to know annual cash flow requirements;
- Estimate foreign exchange cost if required to be incurred.

- Work out per capita and per connection cost for the construction phase of the project based on design population, and compare these with norms, if any, laid down by the government or with those for similar projects.
- Work out cost per unit of water produced and distributed and compare these with norms, if any, laid down by the government or with those for similar projects.

(ix) Implementation schedule

Prepare a detailed and realistic implementation schedule for all project components, taking into consideration the stage of preparation of detailed design and drawings, statuary clearances from various departments, additional field investigations required, if any, the time required for preparing tender documents, notice period, processing of tenders, award of works/supply contract, actual construction period, the period required for procurement of material and equipment, testing, trials of individual component and commissioning of the facilities, etc.

If consultants' services are required, the period required for completion of their work should also be estimated.

A detailed CPM/PERT diagram showing the implementation schedule for the whole project, as well as those for each component should be prepared, showing linkages and inter-dependence of various activities. Application of latest project management software systems should be encouraged for efficient project management.

The implementation schedule should also be prepared for support activities such as training, consumers' education, etc., and their linkages with the completion of physical components and commissioning of the project should be established.

(x) Operation and Maintenance of the project

Estimate annual operating costs, considering staff, chemicals, energy, transport, routine maintenance of civil works, maintenance of electrical/mechanical equipment, consumer service, cost towards occupational health and safety including normal cost of replacement of parts, spares, and supervision charges. Annual cost estimates should be prepared for a period of five years from the probable year of commissioning the project, taking into consideration expected output levels and escalation.

Proposal for monitoring and evaluating the project performance with reference to project objectives should be indicated.

(xi) Environmental and social impact

Brief description of the adverse and beneficial impacts of the project may be given covering the following aspects:

Beneficial Impact	Adverse Impact
Ease and convenience in obtaining safe and sufficient water at household levels. Increase in productivity of people in the time saved and internal social alleviation.	Risk of exploiting natural resources by withdrawing surface/ground water. Risk of affecting flora and fauna of surface water stream.
Improvement in public reuse of water in household premises or by water authority.	Effect of disposal of backwash water and sludge from water treatment plant.

Beneficial Impact	Adverse Impact
Effect of construction of storage reservoirs on flood moderation, navigation, ground water table, power generation, etc.	Effects of construction of storage reservoirs on ground water table, down stream flow of the stream, the reservoir bed, etc., and effects on ecology.

3.7.3 Institutional and Financial Aspects

In the long term, project benefits depend at least as much on the organisation responsible for operating and maintaining the project as they do on the organisation which constructs it. Sometimes the same organisations are involved in both stages. Where separate entities are involved in construction and O&M, detailed arrangements for a smooth transition from the construction stage to the operational stage should be explained and a clear implementation plan should be in place.

The financial planning and cash flow will affect the execution, operation, and maintenance of the project. A detailed financial analysis has to be carried out to include funding, revenue, and expenditure for the successful implementation of the project.

(i) Institutional aspects

It is necessary to examine the capabilities of the organisations that would be entrusted with the responsibility of implementing the project and of operating the same after it is commissioned. The designated organisation(s) must fulfil the requirements in respect of organisational structure, personnel, financial, health and management procedures, so that effective and efficient performance is expected. This can be done by describing the following aspects:

- History of the organisation, its functions, duties and powers, legal basis, organisational chart (present and proposed), relationship between different functional groups of the organisation, and with its regional offices, its relationship with government agencies and other organisations involved in sector development.
- Public relations in general and consumer relations in particular, extension services available to sell new services, facilities for conducting consumer education programmes, stakeholder consultations, Project Affected Persons (PAPs) consultation, and settling complaints.
- System for identification of losses in system and making it good again by rectifying the deficiencies (NRW reduction and control, power factor rectification, etc.)
- Systems for budgeting for capital and recurring expenditure and revenue, accounting of expenditure and revenue, internal and external audit arrangements, inventory management.
- Present positions and actual staff, comments on number and quality of staff in each category, ratio of staff proposed for maintenance and operation of the project to the number of people served, salary ranges of the staff and their comparison with those of other public sector employees or private sector employees.
- Staff requirement (category-wise) for operating the project immediately after commissioning, future requirements, policies regarding staff training, facilities available for training.
- Actual tariffs for the last five years, present tariff, tariff proposed after the project is commissioned, its structures, internal and external subsidies, the procedure required to be followed/to adopt, new tariff, expected tariff and revenues in future years, proposal to meet shortage in revenue accruals.

- Prepare annual financial statements (income statements. balance sheets and cash flows) for the project operating agency, for five years after the project is commissioned, explain all basic assumptions for the financial forecast and the terms and conditions of tapping financial sources, demonstrate ability to cover all operating and maintenance expenditure and loan repayment, workout rate of return on net fixed assets and the internal financial rate of return of the project.

(ii) Financing Plan

Identify all sources of funds for implementation of the project, indicating year-by-year requirements from these sources, to meet expenditure as planned for committing the project as per schedule; state how interest during construction will be paid, or whether it will be capitalised and provided for in the loan; explain the procedures involved in obtaining funds from the various sources.

3.7.4 Record Keeping

Record keeping has to be an integral part of any water supply utility and must maintain all the records (including historical records) of the drawings, investigation reports, project reports, analysis carried out, as-built drawings, O&M records, records of hazards, events, etc. With the advent of digital technology, all the records have to be stored in a digital format and made available to the officer in-charge for designing, maintaining, and further planning of water supply system. A dedicated record keeping personnel has to be appointed who takes the ownership of maintaining and up-keeping/ updating of records of water supply systems.

3.7.5 Conclusions and Recommendations

This section should discuss the justification of the project, in terms of its objectives, “Drink from Tap” with 24×7 pressurised water supply system, achieving SLBs, cost-effectiveness, affordability, the willingness of the beneficiaries to pay for services, and the effect of not proceeding with the project. Issues, which are likely to adversely affect project implementation and operation, should be outlined and ways of tackling the same should be suggested. Confirmation of sustainability of water source from the concerned authority such as central/state groundwater authority/central water commission/state water resource authorities may be received. Effect of changes in the assumptions made for developing the project, on the project implementation period, benefits, tariff, costs, demand, etc., should be mentioned.

Definite recommendations should be made regarding time-bound actions to be taken by the various agencies, including advance action which may be taken by the lead agency pending approval and financing of the project.

3.8 Detailed Project Report (DPR)

The DPR stage arrives once the project feasibility is assured and the authorities approve the pre-feasibility/feasibility report. The fundamentals, viz., water availability, sustainability, capacity to execute and implement as well as O&M, are established in the feasibility report, however, need to be reconfirmed and re-assessed in detail.

Thereafter, a detailed survey and investigation to assess the sites and existing infrastructure is carried out based on which specific requirements are identified for achieving the desired SLB and then followed by detailed engineering and design of all the components including environmental and social impact assessment. GIS-based survey planning and hydraulic design of water supply systems should be carried out to ease in O&M of the systems. Based on these details, cost estimates are prepared which also incorporate costs of land acquisition, actual items in execution of work, safeguards, and mitigation measures. A detailed financial analysis is carried out covering all the aspects of revenue, and expenditure to ensure financial sustainability of the proposed water assets being created and adhering to various government policies being enforced from time to time. These aspects have been discussed and explained in various chapters of this manual.

The DPR has to be prepared as per DPR template (including checklist) made available by CPHEEO from time to time. The sections can be;

(a) Executive summary
(b) Background of project
(c) The existing and proposed project, baseline parameters and the proposed Key Performance Indicators (KPI)
(d) Survey and investigations
(e) Specific requirements of the project
(f) GIS-based detailed design of various components
(g) Environmental and social impact assessment
(h) Detailed cost estimate based on latest schedule of rates which should be updated every year for every state/UT (for each region in the state).
(i) Specifications for various Items
(j) Financial planning
(k) Conclusion and recommendations
(l) Checklist for “Drink from Tap” with 24×7 pressurised water supply system project

CHAPTER 4: PLANNING AND DEVELOPMENT OF WATER SOURCES

4.1 Introduction

Water occurs in nature in all its three forms, solid, liquid, and gaseous, and in various degrees of motion. Formation and movement of clouds, rain, snowfall, stream, and groundwater flow are some of the examples of dynamic movement of water. These dynamic formations of water relate to Earth in various kinds of natural sources of water as described below.

Water Resources Management (WRM) is defined by the World Bank (2019) as the “process of planning, developing, and managing water resources, in terms of both water quantity and quality, across all water uses, wherein planning and development of water source is crucial”.

4.2 Types of Water Sources

The origin of all sources of water on land is rainfall/snowfall. Water can be collected as it falls as rain before it reaches the ground, as surface water when it flows over the grounds in rivers or streams, as pooled/stored water in lakes, reservoirs, or ponds, as groundwater when it percolates into the ground and flows as groundwater, or from the sea into which it finally flows. With the advent of modern treatment technologies, recycled water is also a potential source. The quality of the water varies according to the source as well as the medium through which it flows.

Summer monsoon precipitation is the lifeline of India. The *isohyet* map of India is shown in Figure 4.1. The country receives approximately 4,080 billion cubic metres (BCM) of average annual precipitation including snowfall, out of which 3,000 BCM is available during the summer monsoon season. About 50% of the total precipitation (i.e., about 2,000 BCM) flows into rivers. However, due to various constraints of topography and uneven distribution of precipitation over space and time, only about 1128 BCM of the total annual water potential based on surface and ground waters, can be put to beneficial use. This can be achieved from 690 BCM of utilisable surface water and 438 BCM through groundwater. The average assessed per capita water availability in the year 2011 was 1588 $m^{3,}$ which was reduced to 1486 m^3 in 2021. Per capita water availability is further expected to be reduced to 1191 m^3 by 2050.

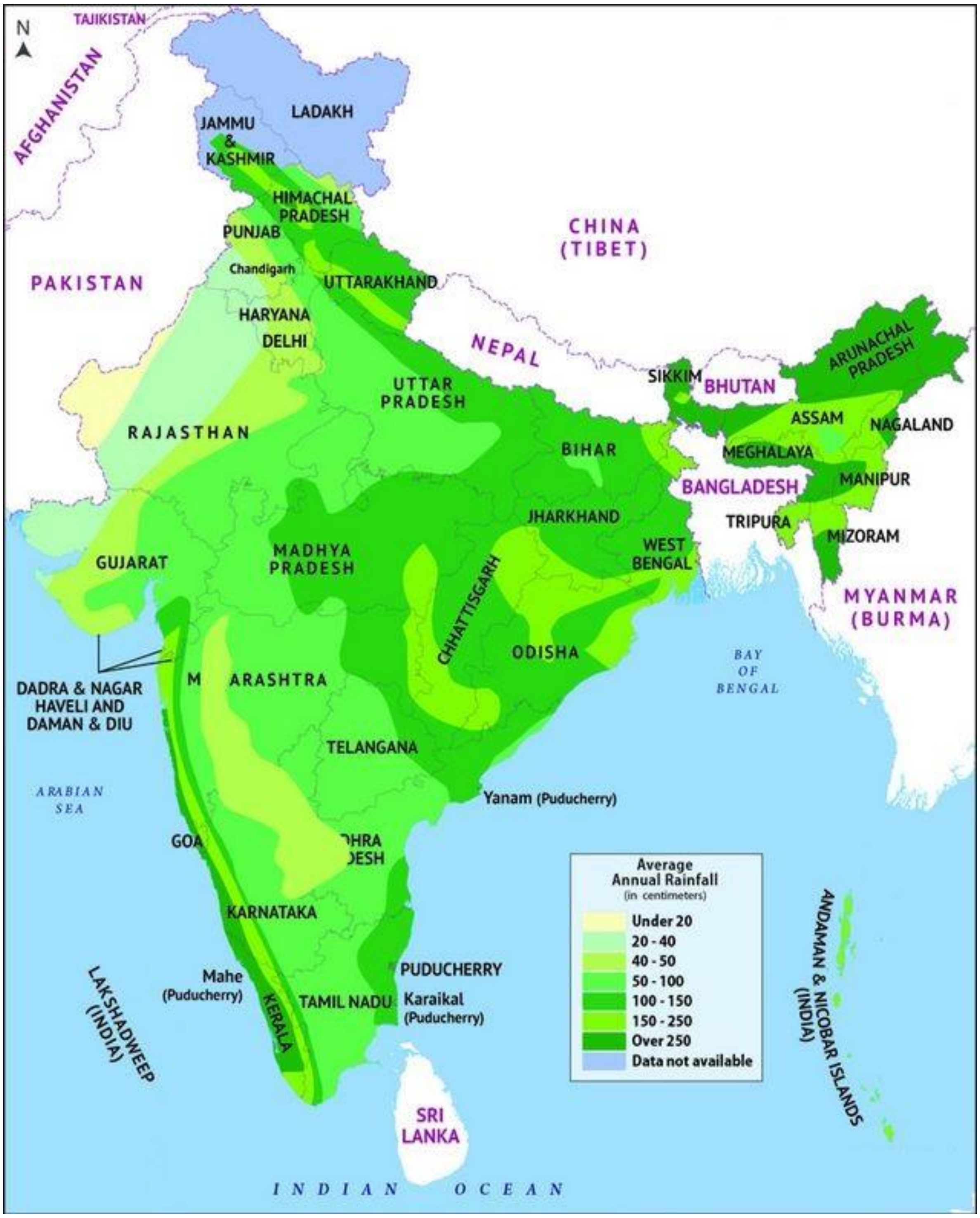


Figure 4.1: Isohyet Map of India with average Annual rainfall in cm

4.2.1 Surface Water Sources

Surface water sources include different water bodies such as rivers, lakes/ponds, springs, tanks, reservoirs, and seawater. India has been divided into 20 river basins as per the report of Central Water Commission (CWC; 2020). The mean annual flow in all the river basins in India is estimated as 1999.2 BCM. Out of this about 35%, i.e., 690 BCM can be put to beneficial uses. The surface water is available in the following forms:

(a) **Natural Quiescent Waters as in Lakes and Ponds:** These waters would be more uniform in quality than water from flowing streams. Long storage permits sedimentation of suspended matter, bleaching of colour and the removal of bacteria depending on the trophic state of lakes. Self-purification which is an inherent property of water to purify itself is usually less complete in smaller lakes than in larger ones. Deep lakes are also subject to periodic overturns which bring about a temporary stirring up of bottom sediment. If the catchment is protected and geomorphologically stable, the stored water may not require any treatment other than disinfection.

(b) **Artificial Waters as in Impounding Reservoir:** Impounding reservoirs formed by hydraulic structures built across river valleys are subject more or less to the same conditions as natural lakes and ponds. While top layers of water are prone to develop algae, bottom layers of water may be high in turbidity, carbon dioxide, iron, manganese and, on occasions, hydrogen sulphide. Soil stripping before impounding the water would reduce the impact of organic load as related to nutrient load and eutrophic state that affects water quality.

(c) **Flowing Waters as in Rivers, other Natural courses, and Irrigation Canals:** Waters from rivers, streams and canals are generally more variable in quality and less satisfactory than those from lakes and impounded reservoirs. The quality of the water depends upon the character and area of the watershed, its geology and topography, the extent and nature of development, seasonal variations, and weather conditions. Streams from relatively sparsely inhabited watersheds would carry suspended impurities from eroded catchments, organic debris, and mineral salts. Apart from sediments, organic pollutants such as dioxin, halogenated compounds, petroleum hydrocarbon, and dibenzofurans, due to anthropogenic activities, also pollute soil and aquatic environment. Substantial variations in the quality of the water may also occur between the maximum and minimum flows. In populated regions, direct pollution by sewage and industrial wastes may also occur. The natural and man-made pollution results in producing colour, turbidity, tastes, odours, hardness, bacterial, and other micro-organisms in the raw water sources.

(d) **Springs**

Springs become active due to the emergence of groundwater on the surface. Until it emerges out on the surface as a spring, the groundwater carries minerals acquired from the subsurface layers, potentially supplying the nutrients to micro-organisms collected by spring, especially if it flows as a surface stream. Spring water from shallow strata is more likely to be affected by surface pollutions than deep-seated water.

Springs may be either perennial or intermittent. The discharge of a spring depends on the nature and size of catchment, recharge, and leakage through the sub-surface. Their usefulness as sources of water supply depends on the discharge and its variability throughout the year.

Various types of springs exist in different hydro-geological environments. These include Depression Springs, Fault Springs, Karst Springs, Hot Springs, Contact Springs, and Artesian Springs. Springs are the major source of drinking water for hilly areas.

The Water Cycle by which water moves between earth and atmosphere is as shown in Figure 4.2:

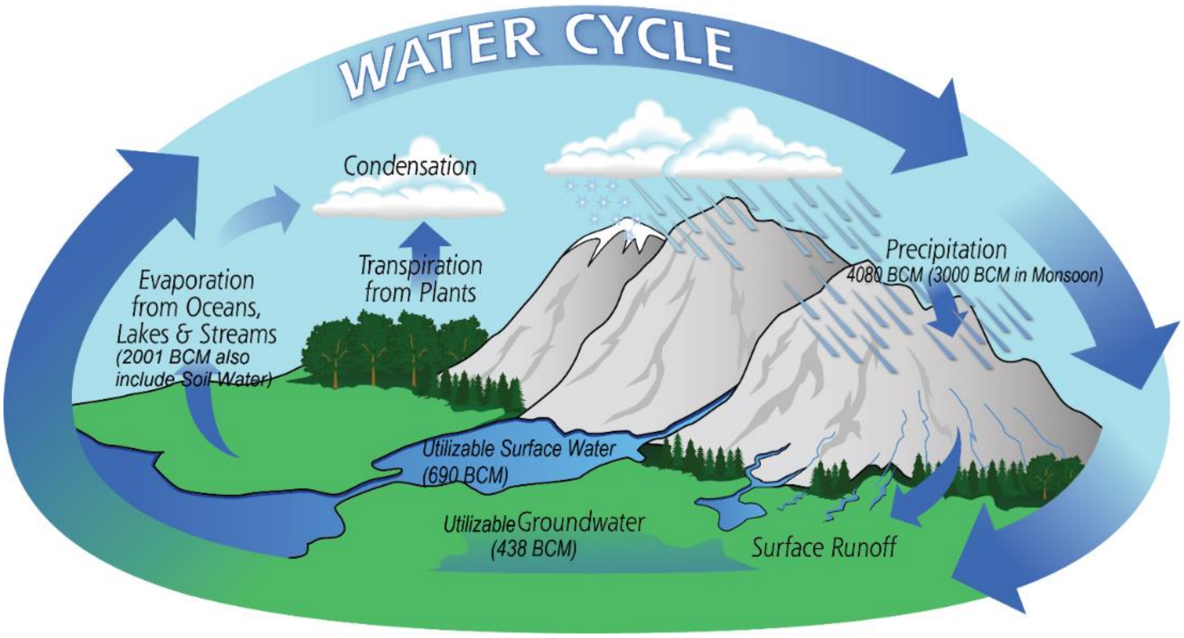


Figure 4.2: Water Cycle

Source: https://gpm.nasa.gov/education/water-cycle

4.2.2 Groundwater

Rainwater percolating into the ground and reaching permeable layers (aquifers) in the zone of saturation constitute as groundwater source. The upper level of zone of saturation is called "water-table". Groundwater is usually free from evaporation losses and its resources are less severely affected by variabilities of rainfall than surface water resources.

As per NITI Aayog, India is the largest groundwater user in the world, with an estimated usage of around 251 BCM per year, i.e., more than a quarter of the global total. With more than 60% of the irrigated agriculture and 85% of the drinking water supplies depend on it. Coupled with growing industrial and urban usage, the groundwater will act as a vital resource.

As per the CGWB assessment of March 2022 (National Compilation of Dynamic Groundwater Resources of India), the total annual groundwater recharge has been assessed as 437.60 BCM. Keeping an allocation for natural discharge, the annual extractable groundwater resource works out as 398.08 BCM. The total annual groundwater extraction (as on 2022) has been assessed as 239.16 BCM. The average stage of groundwater extraction for the country as a whole works out to be about 60.08%.

The extraction of groundwater for various uses in different parts of the country is not uniform. Out of the total 7089 assessment units (Blocks/Districts/Mandals/ Talukas/ Firkas) in the country, 1006 units in various states (14%) have been categorised as "Over Exploited". A total of 260 (4%) assessment units have been categorised as "Critical". There are 885 "Semi-Critical" units (12%) and 4780 (67%) assessment units have been categorised as "Safe". Apart from this, there are 158 assessment units (2%), which have been categorised as "Saline" as major part of the groundwater in the associated aquifers is brackish or saline.

Categorisation based on status of groundwater quantity is defined by stage of groundwater extraction as given below:			
Stage of Groundwater extraction	**Category**	**Status of assessment units**	
		Nos.	**%**
≤ 70%	Safe	4780	67
> 70% & ≤ 90%	Semi-Critical	885	12.5
< 90% & ≥ 100%	Critical	260	4
< 100%	Over-exploited	1006	14
Source: 2022-11-11- GWRA 2022.pdf			

In comparison to 2020 assessment, the total annual groundwater recharge has increased from 436 to 437.6 BCM, where major increase is noticed in the States of Bihar, Telangana, Andhra Pradesh, Tamil Nadu, Arunachal Pradesh, Odisha, and Gujarat. The groundwater extraction has marginally decreased from 244.92 to 239.16 BCM. The overall stage of groundwater extraction has marginally decreased from 61.6% to 60.08%.

The water, as it seeps down, comes in contact with organic and inorganic substances during its passage through the ground and acquires chemical characteristics representative of the strata it passes through.

Generally, groundwater is clear and colourless but is harder than the surface water of the region in which it occurs. In limestone formations, groundwater is very hard, tends to form deposits in pipes, and is relatively non-corrosive. In granite formations groundwater is soft, low in dissolved minerals, relatively high in free carbon dioxide, and is actively corrosive. Bacterially, groundwater is much better than surface water except where subsurface pollution exists. The pollutants include biological as well as chemical components such as pollens, virus, bacteria, household pets' saliva, household dust, arsenic, uranium.

Shallow Aquifer: The upper unconfined aquifers are branded as shallow aquifer which bear at least two water bearing zones down to about 50 m to 70 m depth. Shallow aquifer is a source of dug wells and shallow bore wells. Shallow groundwater is a condition where seasonal high groundwater table or saturated soil is less than 3 m from land surface. Shallow aquifers are easily rechargeable and relatively easily contaminated.

Deep Aquifer: Deep confined aquifers occur below shallow unconfined aquifers separated by impervious layers. Deep confined aquifers are those located beyond 100 m depth below ground level. Deep aquifers bear relatively deeper water level. Deep aquifers also experience significant lag time in their response to climatic variations in comparison to shallow aquifers. Deep aquifers are normally recharged through injection well bores commonly known as aquifer storage and recovery (ASR) wells where treated water is used for recharge.

Well Water: The proper siting and design of a well depends upon a region's geology, climate, distance to stream, and relation to area of recharge and discharge and the topography. To protect the water supply, wells should be located as far as possible away from potential sources of contamination.

4.2.3 Seawater

Though this source is plentiful, it is difficult to economically extract and generate potable water because it contains 3.5% of salts in solution, which involves costly treatment. Offshore waters of the oceans and seas have a salt concentration of 33,000 to 37,000 mg/L of dissolved solids including 19,000 mg/L of chloride, 10,600 mg/L of sodium, 1,270 mg/L of magnesium, 880 mg/L of sulphur, 400 mg/L of Calcium, 380 mg/L of potassium, 65 mg/L of Bromine, 28 mg/L of carbon, 13 mg/L of strontium, 4.6 mg/L of boron. Desalting or de-mineralising processes involve separation of salt or water from saline waters. This is a costly process and has to be adopted in places where seawater is the only source available and potable water has to be obtained from it, such as in ships on the high seas or a place where an industry has to be set up and there is no other source of supply.

4.2.4 Wastewater Reclamation and Reuse

Considering the shortage of water in many urban/peri-urban areas, Government of India (GoI) is encouraging ULBs to utilise their treated sewage water for non-potable reuse (e.g., for recharging groundwater after giving the necessary levels of treatment to suit the nature of use) and non-potable reuse applications (e.g., water for cooling, flushing, lawns, agriculture, horticulture, parks, fire-fighting, and for certain industrial purposes).The Atal Mission for Rejuvenation and Urban Transformation 2.0 (AMRUT 2.0) envisages major reforms for recycle of treated used water to meet at least 20% of total city water demand and 40% for industrial water demand at state level.

4.3 National Water Policy (2012)

Ministry of Jal Shakti, Government of India formulated the National Water Policy (2012) to govern the planning and development of water sources and their optimum utilisation. It has recognised the need for according the highest priority to the drinking water supply. That is why, currently, all water resources projects are planned, designed, and constructed with domestic water supply component to meet the requirements of nearby villages, towns, and cities.

Objective of the National Water Policy is:

- to take cognisance of the existing situation;
- to propose a framework for creation of a system of laws and institutions;
- to prepare a plan of action with a unified national perspective;
- to base planning on river basins and river sub-basins.

The highlights of the National Water Policy (2012) pertaining to drinking water supply are as follows:

- It states that the present scenario of water resources and their management in India has given rise to several concerns; one of them is that access to safe water for drinking and other domestic needs.
- Water is required for domestic purposes along with other uses. The utilisation of all these diverse uses of water needs to be optimised and an awareness of water as a scarce resource should be fostered.
- Safe water for drinking and sanitation should be considered as pre-emptive needs, followed by high priority allocation for other basic domestic needs including needs of animals, etc. Available water should thus be allocated in a manner to promote its conservation and efficient use.
- There is need to remove the large disparity between the water supply in urban areas and in rural areas.
- Urban and rural domestic water supply should preferably be from surface water in conjunction with groundwater.
- Urban domestic water systems need to collect and publish water accounts and water audit reports indicating leakages and pilferages, which should be reduced taking into consideration the social issues.

- Water pricing ensures its efficient use and conservation. In order to meet equity, efficiency, and economic principles, the water charges should preferably be determined on volumetric basis. Such charges should be reviewed periodically.
- Policy 2012 also envisages that there is need for comprehensive legislation for optimum development of interstate river valleys and to enable the establishment of basin authorities with appropriate powers to plan, manage and regulate the utilisation of water resources in the basins.

4.4 India Water Resource Information System (WRIS)

India WRIS was initiated through a MoU on 3 December 2008, between Central Water Commission (CWC), MoWR (now Ministry of Jal Shakti), and the ISRO, Department of Space. India WRIS provides a single window solution for all water resources data and information on GIS framework. It allows user to access and analyse water data for planning and development of water resources in the context of Integrated Water Resource Management (IWRM). It is a web-based platform in public domain.

India WRIS Web-based GIS has 12 major info systems, 36 sub info-systems including 95 data layers, classified under five major groups:

1) Watershed atlas
2) Administrative layers
3) Water resources projects
4) Thematic layers
5) Environmental data

Major layers developed under India WRIS are basins, watershed, river, waterbody, urban and rural population extents, dams, barrage/weir/anicut, canals, and command boundaries, etc. All unclassified data of CWC and CGWB is available in the portal for free download. The information system has dedicated sub-info system of various components of surface water, groundwater, hydro-met observations, water quality, snow cover, inter-basin transfer links, socio-economic parameters, as well as infrastructural and administrative layers.

Customised maps can be generated using “Create your WRIS Module”. India WRIS Web-GIS has saving/printing capabilities:

WRIS Website:

- Surface water quality sub-info system
- Groundwater quality sub-info system
- Telemetry module
- Reservoir module
- Snow cover/Glacial sub-info system

For detailed description about WRIS, reference can be made to https://indiawris.gov.in/wris/#/.

4.5 Water Resource Potential of River Basins

India is blessed with many rivers. Twelve of them are classified as major rivers whose total catchment area is 252.8 million hectares (MHa). Of the major rivers, the Ganga-Brahmaputra Meghna system is the biggest with a catchment area of about 110 MHa which is more than 43 percent of the catchment area of all the major rivers in the country. The other major rivers with catchment area more than 10 MHa are Indus (32.1 MHa), Godavari (31.3 MHa), Krishna, (25.9 MHa.) and Mahanadi (14.2

MHa). The catchment area of medium rivers is about 25 MHa and Subernarekha with 1.9 MHa catchment area is the largest river among the medium rivers in the country.

Besides major and medium river systems, the inland water resources include several reservoirs, tanks, ponds, lakes, and brackish water that cover about 17 MHa of area. About 50% of inland water resources are spread over the states of Andhra Pradesh, Gujarat, Karnataka, Odisha, and West Bengal that cover about 7 MHa of area. River basin is recognised as a basic hydrologic unit for planning and development of water resources.

Government of India is contemplating creation of National Interlinking of Rivers Authority (NIRA), the status of which is outlined in the box below:

National River Linking Project (NRLP)

The NRLP programme envisages the transfer of water from water excess basin to water-deficit basin by inter-linking 37 rivers of India by a network of almost 3000 storage dams. Perspective plan was prepared in August 1980 by Ministry of Irrigation (now Ministry of Jal Shakti). Under NRLP, the National Water Development Agency (NWDA) has identified 30 links (16 under peninsular components and 14 under Himalayan components) for preparation of Feasibility Reports. GoI is contemplating creation of National Interlinking of Rivers Authority (NIRA) for planning, investigation, financing, and implementation of the river interlinking projects in the country, and it will replace existing National Water Development Agency (NWDA).

Water resources potential in river basins in India and utilisable surface water resources are shown in Table 4.1 and Figure 4.3. Water demand for various sectors from 2010 to 2050 is given in Table 4.2.

Figure 4.3: Various River Basins in India

(Source: India WRIS Database, National Water Informatics Centre, Ministry of Jal Shakti, Department of WR, RO & GR)

Table 4.1: Surface Water Resource Potential of River Basins of India (CWC, 2020)

Sl. No.	River Basin	Catchment area (Sq. Km)	Average Water Resources Potential (BCM)	Utilisable Water Resources (BCM)
1.	*2.*	*3.*	*4.*	*5.*
1	Indus	317,708	45.53	46
2	Ganga-Brahmaputra Meghna			

Sl. No.	River Basin	Catchment area (Sq. Km)	Average Water Resources Potential (BCM)	Utilisable Water Resources (BCM)
1.	*2.*	*3.*	*4.*	*5.*
	(a) Ganga	838,803	509.92	250
	(b) Brahmaputra	193,252	527.28	24
	(c) Barak and others	86,335	86.67	----
3	Godavari	312,150	117.74	76.3
4	Krishna	259,439	89.04	58
5	Cauvery	85,167	27.67	19
6	Subarnarekha	26,804	15.05	6.8
7	Brahmani-Baitarni	53,902	35.65	18.3
8	Mahanadi	144,905	73	50
9	Pennar	54,905	11.02	6.9
10	Mahi	39,566	14.96	3.1
11	Sabarmati	31,901	12.96	1.9
12	Narmada	96,660	58.21	34.5
13	Tapi	65,806	26.24	14.5
14	West flowing rivers from Tapi to Tadri	58,360	118.35	11.9
15	West flowing river from Tadri to Kanyakumari	54,231	119.06	24.3
16	East flowing rivers between Mahanadi and Pennar	82,073	26.41	13.1
17	East flowing rivers between Pennar and Kanyakumari	101,657	26.74	16.5
18	West flowing rivers of Kutch and Saurashtra, including Luni	192,112	26.93	15
19	Area of inland drainage in Rajasthan	144,836	Neglect	------
20	Minor rivers draining into Myanmar (Burma) and Bangladesh	31,382	31.17	------
	Total	3,271,953	1,999.2	690.1

Table 4.2: Water Demand for Different Uses

S. No.	Total Water Requirement for Different Uses (in BCM) by Standing Sub-Committee of M/o Jal Shakti			
	Uses	**Year 2010**	**Year 2025**	**Year 2050**
1.	Irrigation	688	910	1,072
2.	Municipal	56	73	102
3.	Industries	12	23	63
4.	Power (Energy)	5	15	130
5.	Others	52	72	80
	Total	**813**	**1,093**	**1,447**

Source: Water and Related Statistics 2021, Central Water Commission, Department of Water Resources, RD & GR, Ministry of Jal Shakti

Table 4.3: Demand and Supply Deficit Data

S. No.	Demand And Supply Deficit Data			
	Uses	**Supply 2020 (BCM)**	**Demand 2050 (BCM)**	**Deficit (BCM)**
1.	Irrigation	540	1,072	532
2.	Municipal	45	102	57
3.	Industries	40	63	23
4.	Power (Energy)	25	130	105
5.	Others	10	80	70
	Total	**660**	**1,447**	**787**

Source: Water Statistics, CWC 2020

Considering, the current supply capacity of 45 BCM for the municipal water supply use and the demand deficit in year 2050 will be reaching 57 BCM as shown in Table 4.3. This can be met by implementing reforms in water supply sector, viz., recycling and reuse, NRW reduction, use of water efficient fixtures, etc.

4.6 Aspects for Selection of Water Sources

The selection of water source is crucial in planning and designing the water supply system and following aspects for selection of surface water and groundwater sources need to be studied for selection of sustainable water source.

4.6.1 Surface Water

Hydrologic inputs play an important and effective role in the planning of water supply projects. Hydrological studies are required at various stages of the project, such as (a) pre-feasibility stage, (b) stage of preparation of feasibility report, (c) planning and design (DPR) (d) project execution stage, and (e) at project Operation and Maintenance stage.

4.6.1.1 Project Hydrology

It encompasses three aspects as described below:

(i) **Assessment of Water Availability:** The water availability is obtained from national streamflow by deducting the storage from streamflow, which is measured by stream gauges. The assessment of water availability of surface water resources in the river basins is extremely important in all the water resources development/water supply projects, because it addresses not only the requirement of irrigated agriculture but also the needs of other uses such as drinking water supply, industries, and power generation. With growing population, the requirement of drinking water supply is becoming critical. Therefore, in all the water resource development projects, the provision is invariably kept for drinking water supply from the storage reservoir. In line with this, all the storage reservoir projects are planned and designed for 100% dependability to meet the drinking water supply requirement even by curtailing other requirements if needed. However, in the case of irrigation and hydropower projects, the dependability criteria may be 70% and 95% respectively.

(ii) **Estimation of Design Floods and High Flood Level (HFL):** Estimation of the design flood and HFL for the project is important from the angle of safety of the intake structures. Therefore, proper selection of a design flood value is significant. A higher design flood value may result in increasing the cost of the intake structure while a low value of the design flood can increase risk to the intake structure and shortage of water intake flow during low water levels.

4.6.1.2 Sedimentation of Reservoirs

Due to rainfall, run-off, and soil erosion in the catchments, reservoirs carry huge quantities of silt. The sedimentation study is carried out while planning water resources projects to estimate the loss of storage of the reservoir during its lifetime. Normally, the life of the reservoir is considered as 100 years as per guidelines (Working Group report and publication no. 19 of CBIP) framed by CWC and CBIP. For the outlet silt levels, 100 years sediment load is considered and for carrying out the stimulation (testing performance of the scheme) studies, 50 years sediment load is considered. Sedimentation near intake structures and intake channels is a very common but critical issue that has to be addressed in design. Sedimentation study of reservoir is carried out using area reduction methods as mentioned in CBIP manual.

Around 3,700 dams in India will lose 26% of the total storage by 2050 due to accumulation of sediments which can undermine water security, irrigation, and power generation as per study by United Nations. (Source: Annexure I/II; Compendium 1122020.pdf)

(i) **Evaporation from reservoir:** Monthly evaporation from reservoirs based on pan evaporimeter data is required while conducting the reservoir stimulation studies for the project. The evaporation is very substantial in many shallow reservoirs which are acting as source and provision of additional reservation has to be kept in the reservoir storage for the summer period.

(ii) **Sanitary Surveillance:** This survey is a study of the environmental conditions that may affect the fitness of surface water as a source. The survey should be carried out 10 km upstream and downstream of the intake point. The scope of the sanitary survey should include a discerning study of the geological, geophysical, hydrological, climatic, industrial, commercial, agricultural, recreational, and land development factors influencing the water drainage into the source and the surface and subsurface pollutions likely to affect it.

4.6.1.3 Assessment of the Yield and Development of the Source

(i) General

A correct assessment of the capacity of the source (e.g., impounding reservoirs) investigated is necessary to decide on its dependability for the water supply project under consideration. The capacity of flowing streams and natural lakes is decided by the area and nature of the catchment, the amount of rainfall and allied factors.

The safe yield of surface sources is decided by its lowest daily dry weather flow (minimum flow in summer) and by the hydrological and hydrogeological features relevant to each case.

(ii) Factors in Estimation of Yield

The incidence and the intensity of rainfall, the run-off from a given catchment and the actual gauged flows in streams are the main factors in estimating the safe yield from any source. Reliable statistics of the rainfall over representative regions of the catchment area, recorded over a number of years, should be collected wherever available. In order to cover deficiencies in such data, it is desirable that rainfall recording stations are set up over all watersheds as part of a water conservation programme by the State Public Health Engineering Authority.

River gauging records should be collected and studied in regard to such sources under investigation. In respect of estimation of the groundwater resource, aquifer geometries, boundaries and properties, groundwater levels, and surface water-groundwater relationships should be studied.

Surface water yield: Water yield is the estimation of freshwater input (for e.g., rain, snow, and snowmelt) flowing into streams and rivers. Many factors affect water yield, including precipitation, temperature, watershed size and location, and primary water source (i.e., rainfall or snowmelt). Total surface water yield is calculated as sum of surface runoff, groundwater flow, minus the transmission loss.

(iii) Methods for Assessment of Surface Flows

a) Assessing the availability of water at the site

When hydrological observation is carried out at the site of interest and data is available for a sufficiently long period (25 to 30 years or more), the quantity of water at the site can be determined. Current metres are used for velocity measurements which in turn is used for computing the flow of the water in the stream.

b) Assessing the peak discharge (flood) value

The methods generally adopted are as under:

(i) Unit hydrograph method based on rainfall runoff studies (CWC Manual on "Estimation of Design flood: Recommended Procedures" can be referred to);

(ii) Frequency analysis based on rainfall;

(iii) Envelope curves based on observed floods in similar catchments; and

(iv) Empirical formulae based on catchment characteristics.

4.6.2 Assessment of Groundwater Resources

4.6.2.1 Hydraulics of Groundwater Flow

i. General

Groundwater moves from areas of high hydraulic head to areas of low hydraulic head. The rate of flow is proportional to the rate at which head decreases with distance along the path from high head to low. Geologic conditions in the sub-surface control the direction and rate of groundwater movement. Ground water flow is defined in metre/year. Streams flow freely within defined channels while groundwater flows in tortuous path within geological layers.

ii. Directions and Rate

Groundwater flows in response to energy gradient. The amount of potential energy possessed by groundwater is measured by quantity termed as "Hydraulic head". Hydraulic head is the elevation to which water rises in a well. Heads measured in wells tapping unconfined aquifer are used to construct water-table contour map. Total head measured in wells tapping confined aquifer is used to construct potentiometric surface map. The rate of groundwater flow various directly with hydraulic gradient.

Groundwater in unconfined aquifer moves form topographically high areas (recharge) to topographically lower areas (discharge). Between recharge and discharge areas, groundwater flow is always in the direction of hydraulic gradient. For a local-scale flow system, the distance between recharge and discharge is relatively small and for regional scale it is much greater. Lakes, river, and springs are useful in inferring water-table elevation where no wells exist.

iii. Groundwater Table Fluctuations

Groundwater table always fluctuates in response to recharge, stream stage and well pumping. The magnitude and rate of water level fluctuation in a well depend on whether aquifers are confined or unconfined, the amount and intensity of rainfall, pumping rate, soil characteristics and specific yield. Water levels fluctuate seasonally in response to weather factors. Water levels generally decline throughout the summer period and recover during winter period.

4.6.2.2 Methods for Groundwater Prospecting/Aquifer Systems

(i) Remote Sensing

The search for groundwater occurring in pores of the soil, regolith, or bedrocks is greatly aided by remote sensing techniques. It should be understood at the beginning that remote sensing techniques complement and supplement the existing techniques of hydrogeological and geophysical techniques and are not a replacement for these techniques.

For convenience, we can divide the aquifers into two groups:

a. aquifers in alluvial areas; and
b. aquifers in hard rock areas.

a. Aquifers in Alluvial Areas

Most well-sorted sands and gravels are fluvial deposits, either in the form of stream channel deposits and valley-fills or as alluvial fans. Table 4.4 lists the keys to detection of such aquifers on the satellite imagery. Although hydro-geologically significant landforms, etc., can be delineated easily on Landsat images, more details are visible on aerial photographs. In favourable cases, satellite images can be used to select locations for test wells. In other areas, locales can be marked for more detailed ground surveys or through examination of aerial photographs.

Table 4.4: Keys to Detection of Aquifers in Alluvial Areas on Satellite Images

Shape or Form	
S. No.	**Description**
1	Stream valleys; particularly wide, meandering (low gradient) streams.
2	Underfit valleys:
3	Natural levees:
4	Meander loops
5	Meander Scars in lowland; oxbow lakes
6	Braided drainage-channel scars
7	Drainage line offsets; change in drainage pattern;
8	Arc deltas (coarsest materials) and other deltas
9	Cheniera; beach ridges; parabolic dunes
10	Alluvial fans, coalescing fans; bajadas
11	Aligned oblong areas of different natural vegetation representing landlocked bars, spits, dissected beaches, or other coarse and well-drained materials

b. Aquifers in Hard Rock Areas

The groundwater abundance depends on rock type, amount, and intensity of fracturing. The keys to detection of aquifers in hard rock areas are given in Table 4.5. The only space for storage and movement of groundwater in such areas is in fractures enlarged by brecciation, weathering, solution, or corrosion.

Vertical fractures and lineaments represent favourable locations for water wells.

Table 4.5: Keys to Detection of Aquifers in Hard Rock Areas on Satellite Images

Outcropping: Rock Type	
Sl. No.	**Description**
1	Landforms; topographic relief
2	Outcrop patterns;
3	Shape of drainage basins
4	Drainage patterns, density frequency and texture
5	Fracture type and density.
6	Relative abundance, shape, and distribution of lakes
7	Tones and textures (difficult to describe; best determined by study of known examples)
8	Types of native land cover

(ii) GIS Method to Assess Groundwater Resources Potential

Most of the water used for domestic purposes comes from groundwater. Remote sensing, GIS field studies, Digital Elevation Models (DEM) can be fruitfully used in the assessment of groundwater resources.

Evaluating physical and environmental factors controlling groundwater occurrence. The parametric influencing factors include:

- Geomorphology
- Lithology of rock formations
- Land-use/Land cover
- Rainfall
- Slope
- Soil
- Drainage density
- Lineament and rock fracture density

Thematic layers on abovementioned parameters are generated and integrated through RS and GIS techniques. GIS based Multi-Criteria Decision-Making (MCDM) process as a spatial prediction tool can be utilised in exploring potential for groundwater resources of drainage areas. Geomorphology, geology, change, drainage density, slope, lineament density and land-use are influencing factors.

The Analytic Hierarchy Process (AHP) method is used to calculate the weightage of these criteria components. Groundwater potential index values are allocated to research locations and Groundwater Resources Potential Zone Maps (GWPZ) are developed as a result.

The Groundwater Potential Index (GWPI) values are classified as low to very good using multi-decision-making criteria and the Analytic-Hierarchy MCDM-AHP technique. GWPZ-maps are created as groundwater resource potential maps using this technology.

Groundwater rationalisation factors like drainage and lineament density are thought to be more accurate forecasting tools. The findings of such research can be useful in groundwater exploration and development. It may be noteworthy to mention that drainage density in an area is directly proportional to run-off and inversely proportional to permeability of area.

From GIS based groundwater prospection studies, it can be made out that groundwater availability mainly depends on integrating various data-layers of geology, geomorphology, slope drainage and lineament density, rainfall, and land use.

4.6.2.3 Groundwater Resources Assessment

GEC-2015 methodology recommends aquifer-wise estimation of dynamic and static groundwater resources. Groundwater resources are assessed to a depth of 100 m in hard rock areas and 300 m in soft rock areas. Methodology recommends resources estimation once in every three years. For detailed norms of estimation, the CGWA (GEC-2015) Guideline can be referred.

State-wise Groundwater Resource Availability of India: The state-wise assessed groundwater resources of India (2022) are given in Table 4.6.

State-wise depth to water level and distribution of percentage of wells for the period of November 2021 in unconfined aquifer is given in **Annexure 4.1**.

Table 4.6: State wise Groundwater Resources Availability in BCM (2022)

S. No.	States/Union Territories	Groundwater Recharge					Total Natural Discharges	Annual Extractable Groundwater Resource	Current Annual Groundwater				Annual GW Allocation for Domestic	Net Ground Water Availability for future	Stage of Groundwater Extraction (%)
		Monsoon Season		Non-monsoon		Total Annual Groundwater Recharge			Irrigation	Industrial	Domestic	Total			
		Recharge from rainfall	Recharge from other sources	Recharge from rainfall	Recharge from other sources										
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1	Andhra	9.14	9.41	0.91	7.77	27.23	1.36	25.86	6.46	0.16	0.83	7.45	1.09	18.54	28.81
2	Arunachal	1.96	0.94	1.06	0.56	4.52	0.41	4.07	0.02	0.01	0.01	0.03	0.01	4.03	0.79
3	Assam	17.92	1.15	6.52	0.94	26.53	2.56	21.4	2.06	0.01	0.58	2.65	0.62	18.71	12.38
4	Bihar	19.94	7.07	1.14	5	33.15	3.1	30.04	10.01	0.35	3.14	13.5	3.41	16.76	44.94
5	Chhattisgar	8.08	1.8	0.15	2.01	12.04	1.04	11.01	4.62	0.11	0.73	5.46	0.83	5.56	49.58
6	Delhi	0.1388	0.0895	0.0094	0.1728	0.4105	0.0411	0.3695	0.0904	0.0007	0.2716	0.362	0.2878	0.0288	98.1612
7	Goa	0.35	0.02	0	0.04	0.41	0.08	0.33	0.026	0.004	0.048	0.078	0.05	0.25	23.63
8	Gujarat	19	2.63	0	4.83	26.46	1.88	24.58	12.1	0.16	0.82	13.09	1.04	12.18	53.23
9	Haryana	3.15	2.79	0.70	2.83	9.48	0.87	8.61	10.30	0.60	0.65	11.54	0.66	1.04	134.14
10	Himachal	0.6	0.14	0.14	0.15	1.03	0.09	0.94	0.18	0.05	0.12	0.35	0.12	0.59	37.56
11	Jharkhand	4.92	0.45	0.48	0.36	6.21	0.51	5.69	0.93	0.21	0.65	1.78	0.65	3.92	31.35
12	Karnataka	8.83	4.29	1.19	3.43	17.74	1.70	16.04	10.01	0.13	1.09	11.22	1.17	6.34	69.93
13	Kerala	4.25	0.15	0.47	0.87	5.74	0.54	5.19	1.17	0.01	1.55	2.73	2.2	2.18	52.56
14	Madhya	26.87	1.56	0.11	6.69	35.23	2.66	32.58	17.39	0.17	1.69	19.25	1.88	14.21	59.1
15	Maharashtr	20.72	2.43	0.54	8.6	32.29	1.84	30.45	15.29	0.003	1.35	16.65	1.35	14.38	54.68
16	Manipur	0.4	0	0.11	0.01	0.52	0.05	0.47	0.02	0.0002	0.02	0.04	0.02	0.43	7.95
17	Meghalaya	1.29	0.01	0.42	0	1.72	0.17	1.51	0.003	0.0007	0.05	0.05	0.06	1.45	3.55
18	Mizoram	0.19	0	0.03	0	0.22	0.02	0.2	0.000	0.00	0.01	0.01	0.01	0.19	3.96
19	Nagaland	0.36	0.33	0.08	0.02	0.79	0.08	0.71	0.002	0.00002	0.02	0.02	0.02	0.69	2.89
20	Odisha	10.44	2.82	1.81	2.72	17.79	1.44	16.34	5.83	0.16	1.24	7.23	1.37	9.03	44.25
21	Punjab	4.67	9.09	0.72	4.46	18.94	1.87	17.07	26.69	0.16	1.17	28.02	1.19	1.57	165.99
22	Rajasthan	8.71	0.62	0.20	2.61	12.13	1.17	10.96	14.18	0.14	2.23	16.56	2.28	0.87	151.07
23	Sikkim	0.1712	0.0039	0.0956	0.0005	0.2712	0.0271	0.2441	0.0089	0.0022	0.0036	0.014	0.0038	0.2291	6.04
24	Tamil Nadu	7.42	9.76	1.33	2.59	21.11	2.04	19.09	13.68	0.18	0.57	14.43	1.36	6.42	75.59
25	Telangana	7.19	6.66	0.98	6.44	21.27	2.02	19.25	7.257	0.154	0.596	8	3.82	11.23	41.6

S. No.	States/Union Territories	Groundwater Recharge					Total Natural Discharges	Annual Extractable Groundwater Resource	Current Annual Groundwater				Annual GW Allocation for Domestic	Net Ground Water Availability for future	Stage of Groundwater Extraction (%)
		Monsoon Season		Non-monsoon		Total Annual Groundwater Recharge			Irrigation	Industrial	Domestic	Total			
		Recharge from rainfall	Recharge from other sources	Recharge from rainfall	Recharge from other sources										
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
26	Tripura	0.81	0.06	0.22	0.22	1.31	0.25	1.06	0.02	0.0007	0.08	0.10	0.09	0.96	9.70
27	Uttar	35.44	13.96	0.82	21.23	71.45	6.13	65.3	40.72	0.41	5.01	46.14	5.48	19.99	70.66
28	Uttarakhand	1.28	0.31	0.1	0.32	2.01	0.16	1.86	0.63	0.12	0.15	0.89	0.15	0.96	48.04
29	West	15.46	1.65	3.04	3.46	23.61	2.19	21.42	8.38	0.14	1.54	10.07	1.76	11.29	47.01
30	Andaman and Nicobar	0.2979	0.0002	0.3203	0.0001	0.6185	0.0618	0.5566	0.0001	0.001	0.0065	0.0075	0.0069	0.5486	1.35
31	Chandigarh	0.01	0.01	0.00	0.03	0.05	0.01	0.05	0.01	0.002	0.03	0.04	0.03	0.01	80.99
32	Dadra and Nagar	0.06	0.01	0.003	0.02	0.09	0.01	0.08	0.01	0.09	0.01	0.11	0.02	0.01	133.2
	Daman and Diu	0.037	0.001	0.000	0.001	0.038	0.002	0.036	0.003	0.055	0.000	0.057	0.016	0.000	157.927
33	Jammu and Kashmir	1.16	1.94	1.15	0.64	4.90	0.46	4.44	0.31	0.05	0.71	1.07	0.73	3.35	24.18
34	Ladakh	0.01	0.05	0.02	0	0.08	0.01	0.07	0.0003	0.00020	0.03	0.03	0.03	0.04	41.36
35	Lakshadwe	0.01	0	0	0	0.01	0.01	0.01	0	0.00	0	0	0	0	61.6
36	Puducherry	0.06	0.09	0.01	0.04	0.21	0.02	0.19	0.08	0.01	0.05	0.13	0.05	0.05	69.17
	Grand Total	241.35	82.30	24.88	89.07	437.60	36.85	398.08	208.49	3.64	27.05	239.1	33.86	188.03	60.08

Total annual recharge estimated is 437.60 BCM and current total extraction for irrigation, industrial and domestic use comprises of 239.16 BCM.

The estimates are briefly outlined as below:

- Total estimated annual groundwater recharge = 437.60 BCM
- Total annual extractable groundwater resources = 398.08 BCM
- Current annual groundwater extraction for irrigation = 208.49 BCM
- Current annual groundwater extraction for industrial use = 3.64 BCM
- Current annual groundwater extraction for domestic use = 27.05 BCM
- Annual groundwater allocation for domestic use as on 2025 = 33.86 BCM
- Net groundwater availability for future use = 188.03 BCM
- Stage of groundwater extraction (%) = 60.08

Categorisation of Assessment Units

Various groundwater assessment units are categorised as groundwater over-exploited, critical, semi-critical and safe category areas. The status of categorisation of assessment units (Blocks/Talukas, etc.) as of 2022 is given in Table 4.7.

Table 4.7: Categorisation of Blocks/Talukas/Mandals in India (2022)

S. No.	State/Union Territories	Total No. of Assessed Units	Safe		Semi-Critical		Critical		Over-Exploited		Saline	
	States		Nos.	%	Nos.	%	Nos.	%	Nos.	%	Nos.	%
1	Andhra Pradesh	667	598	89.7	19	2.8	5	0.7	6	0.9	39	5.85
2	Arunachal Pradesh	11	11	100.00								
3	Assam	28	27	96.43	1	3.57						
4	Bihar	535	469	87.66	46	8.60	12	2.24	8	1.50		
5	Chhattisgarh	146	116	79.45	24	16.44	6	4.11				
6	Delhi	34	4	11.76	8	23.53	7	20.59	15	44.12		
7	Goa	12	12	100.00								
8	Gujarat	252	189	75.00	20	7.94	7	2.78	23	9.13	13	5.16
9	Haryana	143	36	25.17	9	6.29	10	6.99	88	61.54		
10	Himachal Pradesh	10	10	100.00								
11	Jharkhand	263	241	91.63	11	4.18	6	2.28	5	1.90		
12	Karnataka	234	139	59.40	35	14.96	11	4.70	49	20.94		
13	Kerala	152	122	80.26	27	17.76	3	1.97				
14	Madhya Pradesh	317	226	71.29	60	18.93	5	1.58	26	8.20		
15	Maharashtra	353	272	77.05	62	17.56	7	1.98	11	3.12	1	0.28
16	Manipur	9	9	100.00								

S. No.	State/Union Territories	Total No. of Assessed Units	Safe		Semi-Critical		Critical		Over-Exploited		Saline	
	States		Nos.	%	Nos.	%	Nos.	%	Nos.	%	Nos.	%
17	Meghalaya	12	12	100.00								
18	Mizoram	26	26	100.00								
19	Nagaland	11	11	100.00								
20	Odisha	314	300	95.54	8	2.55					6	1.91
21	Punjab	153	17	11.11	15	9.80	4	2.61	117	76.47		
22	Rajasthan	302	38	12.58	20	6.62	22	7.28	219	72.52	3	0.99
23	Sikkim	6	6	100.00								
24	Tamil Nadu	1166	463	39.71	231	19.81	78	6.69	360	30.87	34	2.92
25	Telangana	594	494	83.00	80	13.60	7	1.20	13	2.20		
26	Tripura	59	59	100.00								
27	Uttar Pradesh	836	557	66.63	169	20.22	47	5.62	63	7.54		
28	Uttarakhand	18	14	77.78	4	22.22						
29	West Bengal	345	232	67.25	31	8.99	22	6.38			60	17.39
30	Andaman and Nicobar	36	35	97.22							1	2.78
31	Chandigarh	1			1	100.00						
32	Dadra and Nagar Haveli	1							1	100.00		
33	Daman and Diu	2							2	100.00		
34	Jammu and Kashmir	20	19	95.00	1	5.00						
35	Ladakh	8	7	87.50	1	12.50						
36	Lakshadweep	9	7	77.78	2	22.22						
37	Puducherry	4	2	50.00			1	25.00			1	25.00
	Grand Total	7089	4780	67.43	885	12.48	260	3.67	1006	14.19	158	2.23

Note:
Blocks – Bihar, Chhattisgarh, Haryana, Jharkhand, Kerala, Madhya Pradesh, Manipur, Mizoram, Odisha, Punjab, Rajasthan, Tripura, Uttar Pradesh, Uttarakhand, West Bengal
Taluks – Goa, Gujarat, Karnataka, Maharashtra
Mandals – Andhra Pradesh, Telangana
District – Arunachal Pradesh, Assam, Meghalaya, Nagaland, Sikkim, Dadra & Nagar Haveli, Daman and Diu, Jammu and Kashmir
Valley – Himachal Pradesh, Ladakh
Islands – Andaman & Nicobar, Lakshadweep
Firka – Tamil Nadu
Region – Puducherry
UT – Chandigarh

Based on groundwater resource assessment and categorisation of areas, it may be made out that 14% of the groundwater assessed units belong to overexploited categories and 67% are categorised as safe category areas (Blocks/Talukas) and 2.2% of assessed units are categorise as saline category areas occurring in different district of various states of the country.

I. Groundwater Quality Monitoring

Groundwater quality is being monitored by Central groundwater board once a year through a network of 15,000 observation wells located all over the country and is aimed at generating background data of different chemical constituents in groundwater on a regional scale.

Main groundwater quality problems in India are given in Table 4.8:

Table 4.8: Groundwater Quality Problems in India

Quality Problem	Permissible Limit	States
Inland salinity	EC value of groundwater is greater than 1,000 mili-Siemens/cm (Unit based on the name of the scientist) at 25 ºC making water non-potable.	Inland groundwater salinity is present in arid and semi-arid regions of Rajasthan, Punjab, Haryana, Gujarat, Uttar Pradesh, Delhi, Andhra Pradesh, Maharashtra, Karnataka, and Tamil Nadu. In some areas of Rajasthan and Gujarat, groundwater salinity is so high that the well waters are directly being used for salt manufacturing by solar evaporation.
Fluoride	Level beyond permissible limit (>1.5mg/L)	221 districts covering 19 states of Andhra Pradesh, Assam, Bihar, Chhattisgarh, Delhi, Gujarat, J&K, Jharkhand, Karnataka, Kerala, Madhya Pradesh, Maharashtra, Odisha, Punjab, Rajasthan, Tamil Nadu, Uttar Pradesh, and West Bengal
Arsenic	Level beyond permissible limit of 0.05 mg/L	86 districts covering 10 states of Assam, Bihar, Jharkhand, Chhattisgarh, Haryana, Karnataka, Manipur, Punjab, Uttar Pradesh, and West Bengal
Iron	High concentration of iron (>1.0mg/L)	22 states of Andhra Pradesh, Assam, Bihar, Chhattisgarh, Goa, Gujarat, Haryana, J&K, Jharkhand, Kerala, Karnataka, Madhya Pradesh, Maharashtra, Manipur, Meghalaya, Odisha, Punjab, Rajasthan, Tamil Nadu, Tripura, Uttar Pradesh, West Bengal and UT of Andaman and Nicobar
Nitrate	Level beyond the permissible limit of 45mg/L	423 districts in 23 states and UTs and mostly from the states of Madhya Pradesh and Uttar Pradesh

II. Regulation and Control of Development and Management of Groundwater

A Central Groundwater Authority (CGWA) has been constituted under section 3(3) of the Environment (Protection) Act 1986 to regulate and control development and management of groundwater resources in the country. Similarly, each state should assign a regulator to regulate and control development and management of groundwater resources in the state. Powers and functions basically include regulation and control, management, and development of groundwater and to issue necessary regulatory directions for the purpose.

Some states have come out with their own groundwater abstraction guidelines and have followed the structure of the groundwater model bill 1970/2005. Wherever states/UTs guidelines are inconsistent with CGWA guidelines, the provision of CGWA guideline will prevail as per CGWA, Ministry of Jal Shakti Notification dated 2020. Sates/UTS are at liberty to suggest additional conditions/criteria based on local hydrogeological situations which shall be reviewed by CGWA/Ministry of Jal Shakti, GoI, before acceptance.

Ministry of Jal Shakti, (CGWA) Notification, dated 24 September 2020.

In pursuance of the directions of Hon'ble National Green Tribunal (NGT), the Department of Water Resources, River Development and Ganga Rejuvenation issued a notification to regulate and control groundwater extraction in the country in supersession to Ministry notification, vide S.O-6140(E), dated 12 December 2018. Guidelines shall continue to be regulated by Central Groundwater Authority (CGWA) by way of issuing "No Objection Certificate" for groundwater extraction to Industries, Infrastructure projects & Mining projects etc. unless specifically exempted.

Groundwater extraction guidelines have been prepared to regulate groundwater extraction and conserve scarce groundwater resources to have sustainable management of water resources in the country.

The entire process of grant of "No Objection Certificate" is online through a web-based application system. Application for issue of NOC is given in **Annexure 4.2**.

III. Aquifer mapping

Aquifer mapping is a scientific technique that uses a combination of geology, geophysical, hydrogeologic, and chemical quality data to determine the aquifer's long-term viability.

The requirement for aquifer mapping originates from the general need for scientific planning in groundwater development under various hydrogeological conditions, as well as the evolution of management strategies for better groundwater governance. Several developed countries have also used the standard UNESCO legend of chart-making to map their groundwater systems. A manual on aquifer mapping using international legend has also been published by the CGWB (MoJS).

The first map of India's hydrogeology, titled "Geohydrological Map of India", was released in 1969 by GSI at a scale of 1:2 million. Following that, CGWB released a wall map –Hydrogeological Map on 1:2 million scale and a 1:5 million scale Hydrogeological Map of India, 1976. CGWB, under the Ministry of Jal Shakti of the Government of India, recently created an Aquifer Map of India on a 1: 250,000 Scale featuring 14 Principal aquifer systems and 42 Major aquifer systems (Manual on Aquifer Mapping). AMRUT 2.0 requires cities to do Aquifer Mapping.

Principal aquifers are regionally extensive aquifers that have high intergranular (alluvial plain and valleys) or fracture permeability (peninsular shield region) and provide high level of water storage and may also supply base-flow to rivers.

Major aquifer systems variously cover 2% to 30% area in the country and include Alluvial aquifers (30% coverage), Basaltic aquifers (17%), Granite-gneiss Aquifer (20%), Sandstone aquifer (8%), and Limestone aquifer (2%).

IV. Aquifer mapping Programme

Aquifer mapping programmes are described as under:

a. **National Project on Aquifer Management (NAQUIM)**

CGWB has implemented National Aquifer Mapping and Management Programmes (NAQUIM) which envisages mapping of aquifers (water bearing formations), their characterisation and development of Aquifer Management plans to facilitate sustainable management of groundwater resource. NAQUIM was initiated in 2012 as part of "Groundwater Management & Regulation" scheme to delineate and characterise the aquifers and develop plans for sustainable groundwater management in the country. The state-wise information is shared with state/UTs. Out of 33 lakh sq. km geographical area of the country, a mappable area of 25 lakh sq. km has been identified by CGWB to be covered under the programmes. So far 15.57 lakh sq. km has been covered in 36 different states and UTs. The entire programme can be viewed referring to website, www.cgwb.gov.in).

Objective of programmes: The objective of programmes include:

- delineation and characterisation of aquifers in three dimensions;
- identification and quantification of issues;
- development of management plans to ensure the sustainability of groundwater resources.

The management plans for each aquifer area are being prepared suggesting various interventions to optimise groundwater withdrawal and identifying aquifer with potable groundwater for drinking purposes. The management plan also includes identification of feasible areas for artificial recharge of groundwater, which can help in arresting declining water levels besides demand side management options including crop diversification and increasing water use efficiency, etc.

Outcome of Aquifer Infiltration Management System include:

1. Maps prepared under NAQUIM programme have been shared with state governments through State Groundwater Co-ordination Committees headed by Principal Secretaries of concerned states. The maps and management plans are helping the state governments in water management and in better decision making.
2. Aquifer mapping programmes have provided detailed information on the aquifer dispositions and their characteristics which are necessary inputs for groundwater management.
3. As a part of NAQUIM programme, the region-specific groundwater management plans have been prepared which suggest appropriate demand and supply side management interventions to improve sustainability of groundwater resources.

b. **Hydro-Geomorphological Maps (Groundwater Prospect Maps)**

Integration of geospatial techniques (Remote Sensing and GIS) for mapping groundwater prospection maps is an important tool in source location, monitoring and conserving groundwater. These include:

- Rock lithology/geology
- Land use/Land cover
- Drainage density and drainage frequency
- Lineament and Fracture density
- Slope (%)

Factor evaluation for groundwater recharge mainly includes drainage density which is directly proportional to watershed run-off and lineament density that is directly proportional to infiltration for use in mapping groundwater potential zones.

The preparation and utilisation of Hydro-geomorphological maps (HGMs) are considered essential using RS-GIS data in facilitating State Govts., using such maps for identifying and siting correct locations for sustainable and productive water wells as well groundwater recharging sites.

National Remote Sensing Agency (NRSA), part of ISRO (Hyderabad), is responsible for the preparation of groundwater prospect maps called “HGMs”. HGMS have been prepared and supplied to various states for use in planned development of urban and rural drinking water sources. A User Manual: “Groundwater prospect Map” has also been prepared by NRSC/ISRO for Ministry of Drinking water and sanitation for use of field level implementing agencies, planners, and monitoring agencies in managing groundwater-based drinking water sources.

c. International Technology on Aquifer Mapping

1. **Mapping Groundwater using Airborne Geophysical System (SKYTEM):** SKYTEM is an innovative and technically advanced airborne geophysical system to map buried aquifers and is acceptable globally as best technique for mapping aquifer water resources. This technology is capable of mapping the top 500 m of earth materials in three dimensions.
2. **Groundwater Exploration and Mapping (GEM) System:** The next generation exploration mapping system and optimisation is the game changing in subsurface intelligence gathering and simulation tool developed by Hydro Nova to explore, measure and map groundwater resources. The system integrates a wide range of latest groundwater observation and detection techniques including Geo-Spatial radar, airborne, seismic, hydro-geophysics as well as exploration drilling and down-hole imaging, providing an unparalleled geographic coverage and geologic versatility.
3. **Satellite based weekly Global Map:** NASA researchers have developed new satellite based global maps of soil-moisture and groundwater wetness conditions. Maps enable visualisation of weekly snapshots of soil moisture/groundwater to get complete forecasts of draught situations.
4. **High Resolution Aquifer Mapping and Management:** CSIR Centre launches Heli-borne surveying technology, a latest technology for groundwater mapping in arid regions.

4.6.3 Coastal Aquifer Systems

The groundwater system that trespasses land-sea boundaries is known as coastal aquifers. Coastal aquifers are sources of fresh water for those who live near the coast. For coastal villages, groundwater is the only source of drinking water, as well as the primary source of water for kettle-hole ponds.

Rainfall is the primary source of fresh water in the coastal aquifers system. All water that enters the aquifer system as recharge eventually makes its way to the sea. The hydrogeological balance between fresh groundwater and surrounding dense saline groundwater controls the position and movement of the boundary between fresh and saline groundwater.

4.6.3.1 Groundwater Table in Coastal Aquifer

The height of water table varies throughout the coastal-aquifer system, where recharge and pumping conditions and hydrogeological framework affect the height and configuration of water table. Schematic diagram of groundwater flow in unconfined coastal aquifers system and groundwater flow patterns in coastal areas is shown in Figure 4.4 and Figure 4.5 respectively:

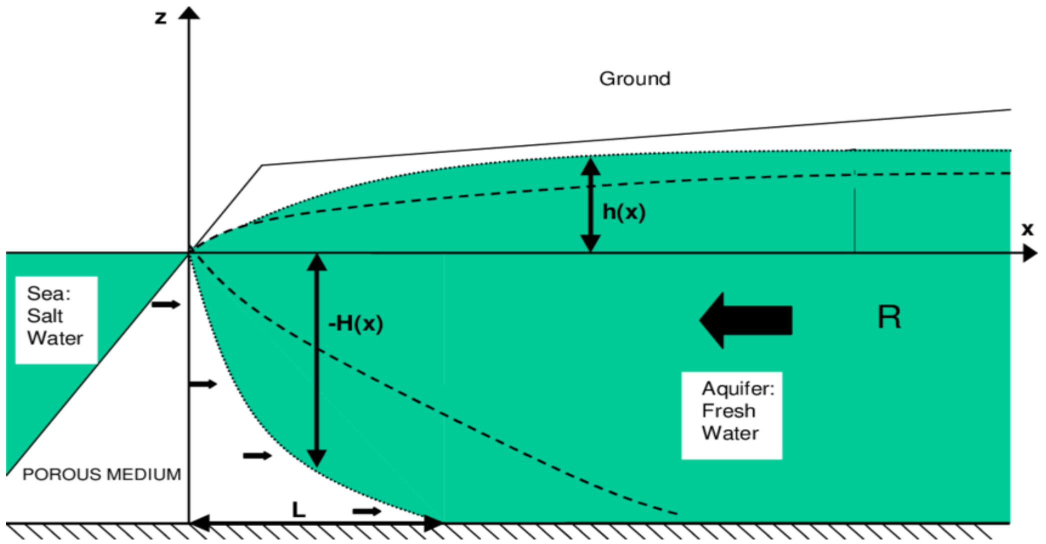


Figure 4.4: Groundwater Flow in Unconfined Coastal Aquifers System
(Source: Encyclopaedia of Ocean Scenarios (Second Edition) 2009)

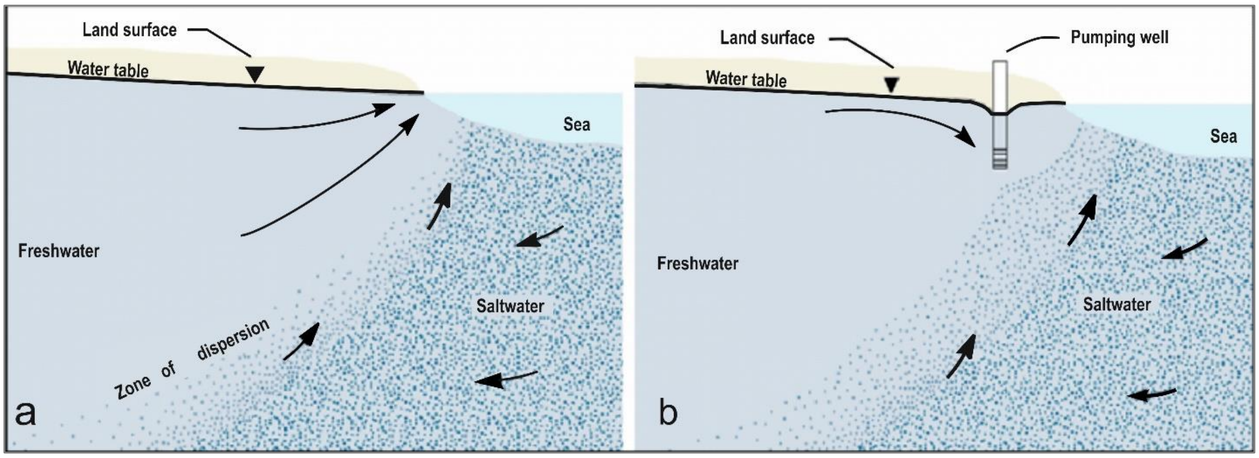


Figure 4.5: Groundwater Flow Patterns in Coastal Areas
(Source: Mary P. Anderson et. Al.: Applied Groundwater Modelling (Second Edition) 2015)

Managing Coastal Aquifer System

The looming problem of saline intrusion and groundwater levels in coastal aquifers must be adequately handled through regular monitoring. The management efforts may include:

(i) constant and regular monitoring of well pumping and movement of fresh and saline water interface;
(ii) determining the cause of brackishness in aquifers using “Isotopic studies”;
(iii) monitoring total influence of saline water intrusion on coastal aquifers using integrated geochemical and geophysical technique and decoding subsurface geological patterns on the line of study done by CGWB in Thiruvallur district, Tamil Nadu;
(iv) improving reduction on over-extraction of groundwater from coastal aquifers through CGWA advised regulatory measures.

4.6.3.2 Groundwater Quality in Coastal Aquifers

The coastline of India covers nine states and one union territory. The status of groundwater quality in coastal aquifers is described as follows. The status and factors affecting the quality of groundwater in coastal aquifers are outlined.

i. Quality along West Coastal Areas

(a) **Kerala:** In Kerala coastal plain, electrical conductivity of shallow groundwater is in the range of 10 to 700 μS/cm. Fluoride content of shallower groundwater is generally less than 0.5 mg/L. The deeper Varkala aquifer yields fresh water with chloride content of 10 and 200 mg/L with higher values occurring around Allepy. Also, the iron content is in the range of 0.1 to 14.0 mg/L. The fluoride content of deeper aquifers is within the range of 0.3 to 2.6 mg/L. The nitrate content in Kuttanad region is within the range of 5 to 17 mg/L. The water of Vaikam aquifers, south of Kuttanad is of calcium carbonate type, whereas in the northern parts, it is of sodium chloride type. Brackish water of Vaikam aquifers has 700 mg/L of chloride and high iodide of about 300 times that of freshwater-seawater mixture.

(b) **Karnataka and Goa:** In coastal plains of Karnataka, water in shallow aquifers, in general, is fresh with electrical conductivity less than 1000 μS/cm, except in localised portions in and around Hangarkatta in Kundapura block of Udupi district where electrical conductivity and chloride value of seawater are recorded as 4230 μS/cm and 980 mg/L respectively.

(c) **Maharashtra:** In coastal districts of Maharashtra, groundwater is alkaline in nature. The groundwater is not highly mineralised. Spatial distribution of electrical conductivity values of groundwater is in the range of 250 to 750 μS/cm between Raigad-Thane belt, whereas it is generally in the range of less than 250 μS/cm in coastal stretch between Raigad and Sindhudurg area. The chloride level of groundwater between Raigad and Sindudurg coastal belt is less than 100 mg/L. The fluoride level of groundwater is generally below 1.5 mg/L in all aquifers in the coastal tract.

(d) **Gujarat:** The Bhawnagar–Una section along Saurashtra coast is affected by seawater ingress and inherit salinity while Madhavpur-Maliya section has the effects of all factors like inherit salinity, seawater ingress, tidal inundation, marshy and seepages and saline alluvium. In coastal part of mainland Gujrat, groundwater is affected by salinity over a limited area. In Kutch area, the groundwater salinity due to ingress is restricted to narrow coastal strip of low-lying Bani plains. Electrical conductivity of water from deep confined aquifers of 100-200 m depth is less than 1000 μS/cm in Basalt, and more than 1500 μS/cm in alluvial/sandstone aquifers.

ii. Quality along East Coastal Areas

(a) **Tamil Nadu:** In situ groundwater salinity problem has been recorded in the following areas:

- Minjur area, north of Chennai city, Chennai district (saline water intrusion problem)
- Thiruvanmiyur-Kovalam tract, southern part of Chennai city (seawater intrusion reported)
- Cuddalore coast: Seawater intrusion and in situ salinity reported
- Ramanathapuram, Nagapattinam, Thanjavur and Tuticorin district (in situ salinity problem)
- Kuttam-Radhapuram area, Tuticorin district (seawater intrusion reported)

In coastal tract of Tamil Nadu and Pondicherry, the location of fresh saline groundwater interface has varied with time due to exploitation of groundwater. In Minjur area (north of

Chennai City) the interface was about 3.5 km inland in 1972. Which has presently moved to about 15 km inland.

(b) Andhra Pradesh: The saline groundwater at moderately deeper levels has been observed due to resident saline seawater. In east Godavari part of coastal area, some improvements in quality of groundwater are reported due to flushing of in situ saline water with continuous irrigation by Godavari canal water. Andhra Pradesh coast was subjected to transgression and regression studies in the past.

(c) Odisha: An area of 8575 sq.km of the coastal districts of Balasore, Bhadrak, Jajpur, Kendrapara, Jagatsinghpuri, Cuttack, Puri and Khurda suffers from groundwater salinity. Saline groundwater in the coastal tract has a width of 15 km in the extreme northeast around Karangasul, 1.5 to 5.0 km in the northern part between Balasore and Kalyani sector and maximum of 75 km in the central part of Mahanadi Delta. The salinity of groundwater is prominent in the deltas of Mahanadi-Brahman, Subharnrekha and Bhurabalang and most prominent salinity groundwater hazard is present in the central part of the coastal tract. Freshwater aquifer overlying the saline water zones occur in Cuttack and Puri districts in parts of Kendrapara, Jagatsinghpuri and Jajpur Districts. The conditions of saline water zones overlying freshwater aquifers exist prominently in Balasore, Bhadrak, Kendrapara, Jagatsinghpuri and Jajpur. Presence of saline water throughout down to explored depth of 600 m is conspicuous in Puri district and in pockets of Kendrapara and Jagatsinghpuri districts. The salinity is also conspicuous in northern part along Karangasul/Chandaneshwar to Chandipur. In Cuttack district, 45 to 55 m thick freshwater aquifers occurring within 90 to 100 m depth is underlain by saline water zone beyond 300 m depth. In Puri district, major part of coastal alluvium suffers from salinity hazard. In Cuttack- Jagatsinghpuri- Kendrapara and Jajpur tract, large areas falling in Rajkanika-Aul-Rajnagar-Pattamadai, Kujang, Mahakalpur, Patkura and Ersama block aquifers down to 60 to 320 m depth are saline to brackish in nature and freshwater aquifers occur below this depth.

(d) West Bengal: Groundwater quality issues of West Bengal include:

- salinity hazards;
- arsenic water pollution;
- industrial pollution;
- high iron in groundwater.

Brackish to saline and freshwater bearing aquifers have been developed in different depth zones in Kolkata Municipal Corporation area, South 24 Parganas and in parts of North 24 Parganas, Haora and Purba Medinipur districts. Kolkata Municipal Corporation Area: Due to lowering of piezometric surface, possibility of ingress of brackish groundwater into freshwater in KMC area exists. Monitoring of piezometers is underway by CGWB. In order to combat salinity problem of Hoogly river water due to tidal effects, fresh groundwater is being withdrawn from deep tube wells located between Mahishadal and Chaitanyapur and is being supplied after mixing with treated surface water. The occurrence of arsenic in groundwater above the permissible limit (more than 0.05 mg/L) has been reported to occur in shallow aquifers in parts of 24 Parganas, North 24 Parganas and Haora Districts. High iron content above permissible limits are found in groundwater in shallow aquifers in South 24 Parganas and Haora districts.

4.6.3.3 Saline Intrusion

Saltwater intrusion is the movement of saline water into freshwater aquifer which results in contamination of drinking water resources. It is indicated by the process of higher concentration of chloride and electric conductivity of groundwater in the area. It is of major concern in coastal aquifers. It is the induced flow of seawater into freshwater aquifer. Saltwater encroaches aquifers when fresh groundwater levels decrease relative to sea level, allowing seawater to flow towards and displace the fresh water inland.

a. High Risk Intrusion Areas

These include areas:

- close to the sea coast;
- where the slope is low to moderate;
- areas with limited source for groundwater recharge;
- areas having high density wells and high rate of pumping for wells;
- areas where static groundwater level is below sea level.

b. Prevention of Intrusion

Following management practices over areas of high risk of saltwater intrusion is needed to be practised.

- Avoid drilling in locations immediately close to the coast e.g., within 50 m of coastline.
- Avoid drilling deep in areas in close proximity to the coast.
- Avoid hydro-fracturing in areas close to coast while developing hard rock wells.
- Close of unusable wells.

c. Controlling saline-water Intrusion

Saltwater intrusion can be controlled by maintaining water balance between water extracted and quality of water recharged into aquifer and creating freshwater mound near sea as well as adopting rainwater harvesting and recharging. Aquifer along coast should not be over pumped to control reduction of pumping depth by low-volume, high-frequency pumping i.e., increase the frequency and reduce the duration of well pumping (“well sipping”) to minimize drawdown in the well and the surrounding aquifer. The best way to control and prevent intrusion is continuously monitoring the depth of water table and water quality of coastal aquifers.

4.7 Pollution Control of Source

4.7.1 Preventing Pollution of Surface Water Sources

Pollution has the potential to harm both the aquatic ecosystem and human health. Pollution has different effects on streams and rivers depending on the type of pollutants.

Potential causes of pollution can be:

- intrusion of seawater into streams;
- sanitation including sewerage, sewage treatment residuals and solid waste;
- disorderly maintenance of sewer outflow;
- erosion and sediment;
- regulatory measures to control water pollution;
- information, Education, and Communication (IEC) activities;
- leachates from the solid waste dump sites;
- untreated effluent from sugarcane and other industries.

Control of pollution requires appropriate infrastructure and management plans.

4.7.2 Preventing Pollution of Groundwater Sources

Water pollution sources and prevention measures are described as below:

(i) Groundwater Contamination: It is a major problem in the country particularly in industrial wells. It is more difficult problem to correct groundwater contamination than surface water contamination. Groundwater is vulnerable both to contamination and unmanaged exploitation. Agriculture, urbanisation, as well as unmanaged groundwater exploitation leads to the degradation of groundwater quality. Also pumping of groundwater result in water table depletion, land subsidence, saline water intrusion and intrusion of poor-quality water from streams. It is because of these problems and issues, the contamination, prevention, and corrective measures are needed.

(ii) Contamination Sources: Contamination or polluting sources are classified as “point source” (e.g., underground storage tank) and “non-point source” (such as agricultural). Contaminating system and sources are outlined as below in Table 4.9.

Table 4.9: Contaminating Sources

System		Contaminating/leak sources
1. Septic tank system	:	Commonly used for disposal of domestic waste and wastewater
2. Soak pit and leach pit	:	Used for disposal of effluent of domestic wastewater. Pit-latrines are also commonly used which may cause on-site contamination.
3. Agricultural activities	:	Use of chemicals and fertilises
4. Solid waste disposal	:	Seeps from landfills
5. Underground storage tank	:	Leaks from underground tanks
6. Spills (Overflows)	:	Spills and leaks at industrial site, military bases are points, gas line station, Highways
7. Mining	:	Coal and metal mining operation areas
8. Salt contamination	:	Due to pumping of groundwater in coastal aquifer regions
9. Underground injection	:	Threat to groundwater from waste disposal via injection wells
10. Abandoned wells	:	Uncapped and unsealed abandoned wells
11. Surface water contamination of Groundwater	:	Due to withdrawal of groundwater near a contaminated river that drains surface water into aquifer and contaminate it (i.e., through induced recharge)

4.7.3 Protection of Groundwater:

Groundwater pollution by human activity normally cannot be totally eliminated, but can be minimised.

i. **Preventive Option:** The best option is prevention which includes determining potential sources of pollution and effectively controlling these framers, homeowners, well-drillers well developers, operators of waste-disposal facilities, gas station attendants, fertiliser dealers and manufactures can help curb pollutants/contamination.

ii. **Control of groundwater pollution:** It should begin with preventive actions instead of clean-up measures. Preventive strategies that can be used include the following:

- Zoning
- Land use planning

- Watershed protection
- Observing rules for waste discharge
- Taking into account hydrogeological, socio-economic, and environmental influences on system

Protection strategies are to emphasise public knowledge and well monitoring, enforcement, and good understanding of hydrogeology of areas.

iii. **Developing Aquifers Protection Plan:** Several strategies are required for a successful groundwater protection plan and several strategies need to be developed at local level and at central level/national level. Local rules can establish protection areas for vulnerable/over-exploited aquifers with the need to protecting towns water supply source-plan or well-head protection plan. Prevention is far less costly than restoring the polluted groundwater. Protection plan should include well-monitoring network and enforcement approach. In the preparation of pollution protection plan, following maps and plans are always an inescapable necessity. These include:

- Aquifer Vulnerability
  DRASTIC method of groundwater vulnerability assessment: A GIS-based DRASTIC model for assessing Aquifer Vulnerability is to be used. This model considers the main hydrologic and geological factors with potential impact on aquifer pollution. DRASTIC acronym stands for:
  D - depth to groundwater
  R - recharge rate
  A - aquifer
  S - soil
  T - topography
  I - impact of Vadose's zone and
  C - hydraulic conductivity
- Groundwater level contour maps showing direction and rate of groundwater movement
- Inventorying wells data
- Measuring non-pumping water level in all wells
- Estimating aquifer parameters using well log and aquifer pump-test data (APT)
- Collecting and analysing the water samples for wells/tube wells
- Assigning responsibilities for implementing the plan by local citizen groups / organisations

4.8 Conservation and Restoration of Water Bodies

India is covered by various types of water bodies which include Lakes, Wetland, Ponds etc. Urban Lakes/water bodies are important elements in the landscape. Lakes have traditionally been serving as source of drinking water, household uses, fishing, and for agriculture, religious and cultural purposes.

Lakes are intrinsic part of ecosystem. Because of their relevance to social benefits, they need to be restored, conserved, managed, and maintained. Every lake has catchment, the area from which water drains into it. Run-off water along with pollutants enter the lakes from these areas.

In view of above, the urban water bodies have to be in the focus and realms of planning and decision-making processes because such water resources, if protected and managed properly, will surely produce great potential to augment water supply at least for non-potable requirements of ever-increasing urban population.

Conservation measures by ULBs. Steps include:

(i) Water bodies should be included in municipal land use records.
(ii) The lake shoreline should be properly fenced to safeguard against encroachment.
(iii) Water bodies should be protected with well-designed inlet and outlet structures.
(iv) Protecting urban water bodies from out fall of domestic and industrial sewage based on CPCB guidelines.
(v) De-silting and cleaning of water bodies be done on regular basis including treatment of their catchments.
(vi) Water quality of water bodies may be monitored on monthly and annual basis by concerned ULBs.
(vii) A water conservation authority should be set up at state level to sustain water bodies by rejuvenating them at ecosystem-based approach.
(viii) Water bodies/pond bodies should be part of storm water management plan of each city.

4.9 Development of Surface Sources

4.9.1 Intakes

An intake is a device or structure placed in a surface water source for withdrawal of water from the source and convey the water to an intake conduit through which it will flow into the water works system. Types of intake structures consist of intake towers, intake barge or jetty, submerged intakes, intake pipes or conduits, intake wells, movable intakes, and shore intakes. Intake structures over the inlet ends of intake conduits are necessary to protect against wave action, floods, navigation, ice, pollution, debris, and other interference with the proper functioning of the intake.

Intake towers are used for large waterworks drawing water from lakes, reservoirs, and rivers in which there is a wide fluctuation in water level and/or a desire to draw water at a depth that will yield the best quality to avoid clogging or for other reasons. A schematic of an intake structure (intake well) is as shown in Figure 4.6.

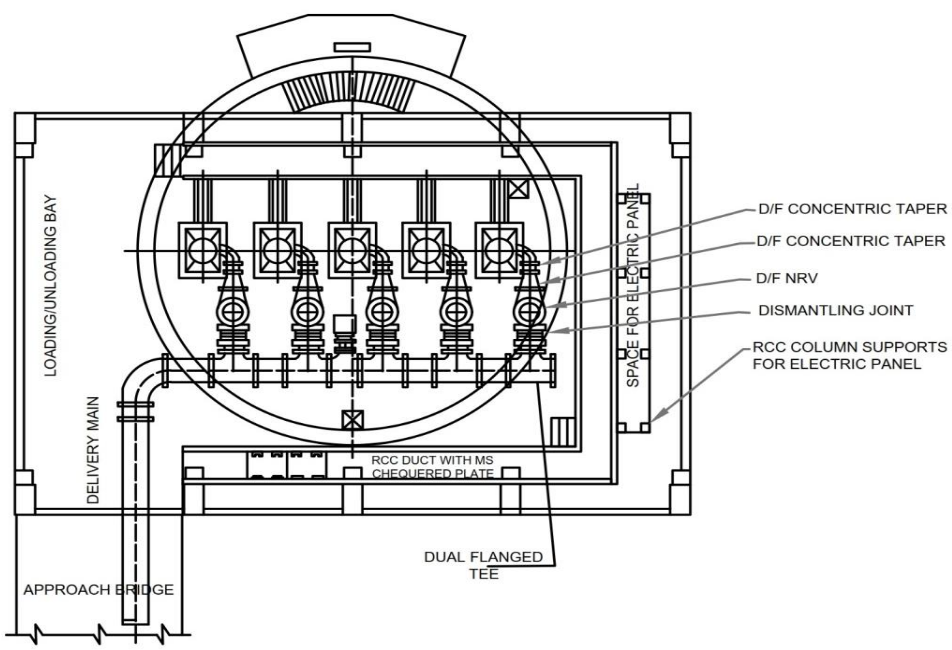


Figure 4.6: Intake Well

Main sources of water intake:

Three main sources of a water intake are:

- surface water (a lake, river, or reservoir);
- groundwater (an aquifer);
- recycled water (reused water).

4.9.1.1 Intake Locating Factors for Surface Water

The following factors should be considered for locating the intake:

(i) The location where the best quality of water is available

(ii) The location shall not be provided at the meandering of the river or stream, i.e., absence of currents that will threaten the safety of the intake

(iii) Above Highest Flood Level (HFL)

(iv) All season road should be constructed for accessibility

An intake in an impounding reservoir should be placed in the deepest part of the reservoir and it should be above the level of maximum accumulated sediments. The deepest portion is ordinarily near the dam, to take full advantage of the reservoir capacity available. Provision for trash arresters (Rose Pieces) at different depths to take advantage of better water quality should be made.

4.9.1.2 Classification of Intake Structure

These are categorised into three categories. Category I comprises submerged and exposed intakes, Category II comprises wet and dry intakes, whereas Category III comprises river, reservoir, canal and lake intakes.

4.9.1.3 Main type of Intakes

1. Impounding reservoir and lake intake
2. River intake
3. Canal intake
4. Fixed jetty intake
5. Intake chamber with removable screens.
6. River bottom intake
7. Floating intake

4.9.1.4 Functions of Intake Structures

Basic Functions are:

- to ensure getting required water;
- to check trash and debris entry along with water and drain;
- to secure entry of water with minimum disturbance;
- to reduce sediment entry.

Description of intakes

i. **Impounding Reservoir and lake intake**
Intake structure is required to withdraw water from surface sources like river, lake, or reservoir. In reservoir it is often built as an integral part of the dam and in others as shoreline structures. Typical intakes are well type circular reinforced cement concrete (RCC) structures with submerged port holes fitted with screens at different levels in staggered manner on the circumference of the well. Location, height, and selection of holes are related to the

characteristic of water, depth of water etc. A control room is constructed on the top of the well to control the mechanism fitted therein to operate the closing and opening of the port holes fitted at different levels. Such intake towers are commonly built for lakes and reservoirs with fluctuating water levels and variation in quality of water with depth.

ii. River intake

River intakes are constructed upstream from point of discharge of community sewage and industrial wastewater. They should be so placed within the river channel as to take advantage of deep water, a stable bottom and better water quality. Streams in which water level during dry months depletes below the normal level of withdrawal a weir may be constructed to raise the level of water.

iii. Canal intake

Canal intake generally consists of masonry or concrete intake chamber of rectangular shape admitting water through coarse screen. A fine screen should be provided over bell mouth entry of the outlet pipe. In case normal flow of canal is not affected, the intake chamber may be constructed inside the canal bank. Preferably lining should be provided to the canal near the intake chamber.

iv. Fixed Jetty intake

The structure is of RCC cast in situ bored piles with Mild Steel (M.S.) liner of design length and thickness (Figure 4.7). The piles are tied with longitudinal and lateral tie beams over which working floor of structure is constructed. There is free passage of water at the inlet of the suction pipes of Vertical Turbine (V.T.) pumps which is subjected to invasion by unwanted floating objects. So, the inlet bell mouth is provided with screen of adequate design to prevent entry of unwanted objects.

Figure 4.7: Fixed Jetty Intake

(Source: https://www.gbcinfrastructure.in/complete-raw-water-intake-plants-projects/)

v. Intake Chamber with removable screens.

This is of RCC construction over the bed of river/lake where suction pipe is placed within the RCC chamber below Lowest Water Level (LWL). Mild steel bar rack is fitted by the upstream side of the chamber followed by removable screen. The screen is useful in preventing entry of unwanted floating objects in surface water. Water is taken out using a suction pipe fitted with an inverted bell mouth. Periodical cleaning of bar rack and removable screen is necessary to keep the intake structure functions adequate for drawl of design quantity of water (Figure 4.8).

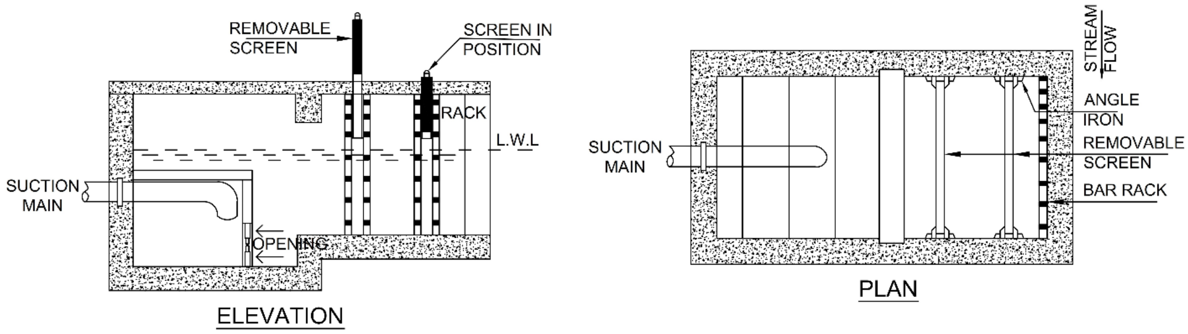


Figure 4.8: Intake with Removable Screen
(Source: http://www.vmmcl.com/projects_intake_structures_palanpur.aspx)

vi. River Bottom intake

River bottom intakes for drinking-water system are used in stream and river, where bed sediment content and bed load are low. Water is extracted through screen over a channel using submersible pump as shown in Figure 4.9. This type is recommended for taking emergency measures for restoration of water during the floods.

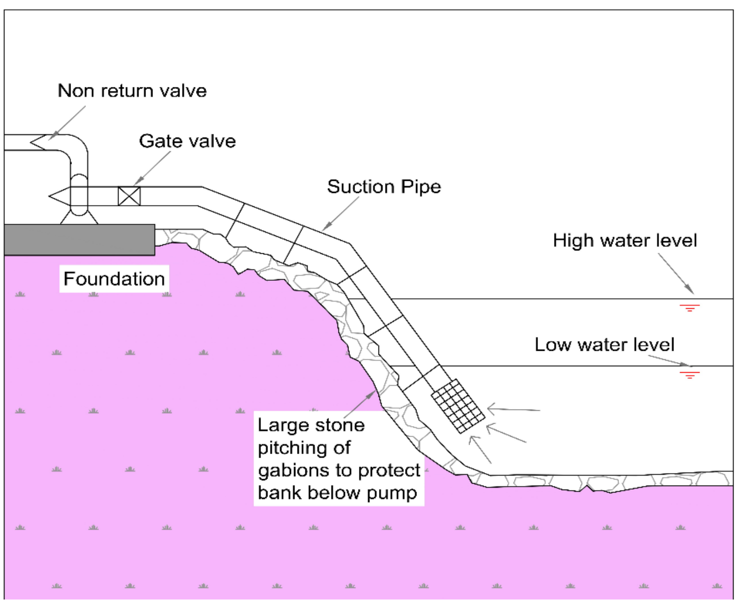


Figure 4.9: River Bottom Intake
(Source: https://repository.lboro.ac.uk/collections/Intakes_rivers_and_weirs/4500617)

vii. Floating Intake:

A floating water intake unit is an investment-friendly solution where there are very large distance changes in the coastal line/riverbank with the vertical water level variation. To ensure continuity in the water receiving unit, it is necessary to reach the deep points of the water basin. This necessitates the use of piled bridges and lift pumps in conjunction with conventional intake well. Floating intake can be built using amphibious floating concrete modules/pontoon that can be placed both on land and on water.

These are for drinking water system that allows water to be abstracted from near the surface of river or duke and avoiding the heavier silt-load. Floating intake system is shown in the Figure 4.10.

The floating intake unit is constructed by placing the floating pump station in a desired area of water and connecting the pipe leading to it by a floating bridge or a floating way line. As an alternative to free surface water intake structures, with this method filtration and sedimentation costs will be significantly reduced as quality of water taken from more stable region and upper elevation is less turbid and transparent.

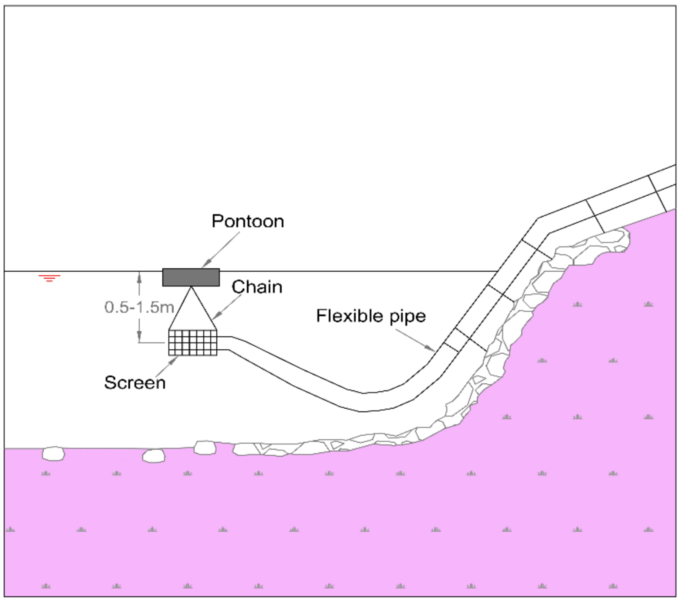


Figure 4.10: Floating Intake

(Source: https://repository.lboro.ac.uk/collections/Intakes_rivers_and_weirs/4500617)

viii. Intake Tower

Intake tower or outlet tower is a vertical tubular structure with one or more openings used for capturing water from reservoirs and conveying it further to water treatment a hydroelectric plant.

ix. Jack Well

A type of intake tower within which the water level is particularly similar to the level of the source of supply. It is known as jack well structure which is used for accumulating water from the surface sources like a river, lake, and reservoir. Since it is under water structure, it is necessary to design accurately.

x. Valve Tower

A valve tower sits above an outlet pipe or tunnel used to transfer water out of the reservoir. It houses equipment and controls for opening and closing gates/valves which enables flow rates of water to be regulated.

4.9.1.5 Design Considerations

The intake structures design should provide for withdrawal of water from more than one level to cope up with seasonal variations of depth of water.

Undermining of foundations due to water currents or overturning pressures, due to deposits of silt against one side of an intake structure, are to be avoided.

The entrance of large objects into the intake pipe is prevented by coarse screen or by obstructions offered by small openings in the crib work placed around the intake pipe. Fine screens for the exclusion of small objects should be placed at an accessible point. The area of the openings in the intake crib should be sufficient to prevent an excessive velocity to avoid carrying settleable matter into the intake pipe.

The conduit for conveying water from the intake should lead to a jack well. For conduits laid under water, standard cast iron or suitable pipe may be used. Larger conduits may be of steel or concrete. A tunnel, although more expensive, makes the safest conduit.

The capacity of the conduit (joining intake well to jack well) and the depth of the jack well should be such that the intake ports to the suction pipes of pumps will not draw air. A velocity of 60 to 90 cm/s in the intake conduit with a lower velocity through the ports will give satisfactory performance. The horizontal cross-sectional area of the jack well should be three to five times the vertical cross-sectional area of the intake conduit. The diameter of the jack well should be selected such that it can accommodate the required pumps along with stand by pumps even for the ultimate stage (30 years after base year).

The intake conduit should be laid on a continuously rising or falling grade to avoid accumulation of air or gas pockets of which would otherwise restrict the capacity of the conduit.

Excessive sand problem: In some rivers, sand is transmitted even to the units of treatment plant making its operation difficult. A suitable sand removable mechanism (detritus tank or plain sedimentation tank) shall be designed to overcome such problem.

4.9.2 Impounding Reservoirs

Impounding reservoir is a basin constructed across the river/stream to store water during excess streamflow and to supply water when the flow of the stream is insufficient to meet the demand for water. For water supply purposes, the reservoir should be full when the rate of streamflow begins to become less than the rate of demand for water. The impounding reservoir can be in the form of dams, Kolhapur Type (KT) weirs, balancing tanks, etc.

Generally, dams constructed by the irrigation department are considered as a source of drinking water supply projects. However, irrigation department constructs dam for which benefit cost ratio is more than one. If this ratio is unsuitable for irrigation purpose, then such left over locations can be considered for non-irrigation purposes such as drinking water and industrial use. As drinking water is the most important for sustenance of life the National Water Policy as considered as top priority. Hence, when there is no alternative source of water, the utility can think of constructing their own impounding storage reservoir at the sites where irrigation department is not contemplating the construction of irrigation dams. Many power generation plants and industries have constructed their own dams for storing of water for their needs.

(a) Choice of Reservoir Site

The suitability of a site must be judged from the following stand points:

(i) Quantity of water available.

(ii) Quality of source.

(iii) Possibility of the construction of a reasonably watertight reservoir.

(iv) Distance of the source from the consumer.

(v) Elevation of the supply.

(b) Physical Considerations

The estimation of the quantity of water of desired quality and a proper location for siting the impounding structure are of primary concern for any water supply scheme. This consists essentially of relating the capacity of the reservoir (and therefore the height of the dam) to the distribution of run-off from the catchment area (i.e., the variations in a steam flow) during a dry period.

(c) Geological Considerations

The decision as to the practicability of dam construction on a particularly favoured site is one which rests largely on geological considerations, viz., the geology of the catchment area, of the reservoir area and of the dam itself. The geological maps should be used to study the nature of the catchment area, the reservoir area, and the dam site.

(d) Site Exploration

The geological investigation should extend to the exploration of the foundations to determine their ability to carry the structure. This will involve the sinking of numerous trial holes or borings in addition to those sunk along the centre line of the dam.

(e) Computation of Storage

Storage can be computed using available scientific methods.

(f) Reservoir Management

i. **Silting:** Loss of capacity due to the deposition of silt in a reservoir will occur and the usefulness of the reservoir will diminish over time. It may be minimised by proper site selection, implement erosion control like afforestation, deploy effective reservoir operation and de-silting works.

   Soil erosion and control are closely related to the silting of reservoirs since without erosion there would be no silting. Erosion prevention methods recommended for soil conservation include proper crop rotation, contour ploughing, terracing, strip cropping, protected drainage channels, check dams, reforestation, fire control, and grazing control.

   Hence it is necessary to provide for silting capacity for all impounding reservoirs, based on studies or data pertaining to similar catchments.

ii. **Evaporation:** Evaporation is of importance in determining the storage requirements and estimating losses from impounding reservoirs, and other open reservoirs. Evaporation from water surface is influenced by temperature, barometric pressure, mean wind velocity, vapour pressure of saturated vapour and vapour pressure of saturated air and dissolved salt content of water. The evaporation loss in storage tanks in India amounts to 2–2.5 m per year.

iii. **Seepage:** Seepage occurs wherever the sides and bottom of the reservoir are sufficiently permeable to permit entrance of water and its discharge through the ground beneath the surrounding hills. Apart from making them impermeable to the extent possible economically, erosion control measures such as proper crop rotation, contour ploughing, terracing, strip cropping, reforestation or afforestation, cultivation of permanent pastures and the prevention of gully formation through the construction of check dams could also be useful on a long-term basis.

iv. **Algal Problems:** Reservoir management comprises of reducing the algal problems and the growth of water hyacinth. Small inflows of water rich in organic matter should be prevented wherever possible instead of allowing them to infect the main body of the water. The water weeds in the reservoir should be controlled by suitable methods such as dragging and underwater cutting. Algicidal measures as described in section 10.2 in Part A of this manual may be adopted to control algae in reservoirs.

4.10 Development of Subsurface Sources

The subsurface sources include springs, wells, and galleries. The wells may be shallow or deep. Shallow wells may be of dug well type, sunk, or built, of the bored type or of the driven type. They are of utility in abstracting limited quantity of water from shallow permeable layers, overlying the first impermeable layer.

Deep wells are wells taken into permeable layers below the first impermeable stratum. They can be of the sunk well type or the bored or drilled type. They are of utility in abstracting comparatively larger supplies from different permeable layers below the first impermeable layer. Because of the longer travel time of groundwater to reach permeable layers below the top impermeable layers, deep wells yield a safer supply than shallow wells.

4.10.1 Spring-shed Management

Springs with significant flow of water (over 20 m^3/h) have usually been developed long ago and are currently used for either irrigated agriculture or human needs, but smaller flows are often overlooked as potential sources of water for livestock consumption in arid and semi-arid regions where a small water flow quickly evaporates if not properly collected and conveyed.

A spring discharge of less than 0.5 m^3/h does not usually show any flow. Water disappears by evaporation and evapotranspiration in the middle of the vegetation which naturally develops around the spring. If properly collected and distributed, the same water could meet the requirements of cattle.

Discharge measurements: A simple and accurate way to determine flow volume of small water supplies 90º V notch. A 'V notch' for determining up to 10 m^3/h can be made from a piece of flat metal measuring 40×25 cm for which a triangular notch with a right angle is cut out. The graduation is to be written on the side of the opening. The position of Graduation (m^3/h) should be as given in Table 4.10:

Table 4.10: Graduation Discharges

Graduation Discharge (m^3/h)	Vertical distance in mm from the bottom of the notch to the Graduation
0.5	19
1	34
2	43
3	52.5
4	59
5	65
10	85

4.10.2 Classification of Wells

The wells are classified according to construction as follows:

(a) Dug wells
(b) Sunk wells
(c) Driven wells
(d) Bored wells
(e) Artesian Wells

(a) Dug Wells

The depth and diameter of drinking wells are decided with reference to the area of seepage to be exposed for intercepting the required yield from the sub-soil layers. Unsafe quality of water may result if care is not taken in the well construction.

The bottom of the well should be at a level sufficiently below the lowest probable summer water table allowing also for an optimum drawdown when water is drawn from the well.

(b) Sunk Wells

Sunk wells depend for their success on the water bearing formations which should be of adequate extent and porosity. The sunk well is only the inter-position of a masonry barrel into such a deposit so as to intercept, as large a quantity of water, as is possible.

The minimum depth of a well is determined by the depth necessary to reach and penetrate, for an optimum distance, the water bearing stratum allowing a margin for dry seasons for storage and for such draw-down as may be necessary to secure the required yield. The method of construction employed depends on the size and depth of the well, characteristics of material to be excavated and quantity of water to be encountered. However, in case of sunk well linings constructed above ground level and then it is sunk in subsoil. It is relatively deep hole than in dug well.

(c) Driven Wells

The shallow tube well, also called a driven well, is sunk in various ways depending upon its size, depth of well and nature of material encountered. The closed end of a driven well comprises a tube of 40 to 100 mm in diameter, closed and pointed at one end and perforated for some distance therefrom.

Such a driven well is adopted for use in soft ground or sand up to a depth of about 25 m and in places where the water is thinly distributed. It is especially useful in prospecting at shallow depths and for temporary supplies. It is useful as a community water stand post in rural area.

(d) Bore Wells

Bore wells are tubular wells drilled into permeable layers to facilitate abstraction of groundwater through suitable strainers inserted into the well extending over the required range or ranges of the-water bearing strata.

Bored wells, useful for obtaining water from shallow as well as deep aquifers, are constructed employing open end tubes, which are sunk by removing the material from the interior, by different methods.

For bored wells, the hydraulic rotary method and the percussion method of drilling such wells through hand soils are popular. For soft soils, the hydraulic jet method, the reverse rotary recirculation method and the sludger method are commonly used.

(i) Well Drilling Methods

Driven wells are constructed by pushing pipe into shallow sand and gravel aquifer to a depth of 6 to 20 m. Most modern wells are drilled using cable tool or rotary drilling equipment.

(ii) Direct Rotary Method

With the hydraulic direct rotary method, drilling is accomplished by rotating suitable tools that cut, chip, and abrade the rock formations into small particles.

Water wells drilled by the hydraulic rotary method generally are cased after reaching the required depth, the complete string of casing being set in one continuous operation.

The hydraulic rotary drilling generally requires large quantity of water which may have to be brought from long distances, if not locally available.

(iii) Percussion Method

In the percussion method of drilling, the hole is bored by the percussion and cutting action of a drilling bit that is alternately raised and dropped. The drill bit, a club like, chisel-edge tool, breaks the formation into small fragments; and the reciprocating motion of the drilling tools mixes the loosened material into a sludge that is removed from the hole at intervals by a bailer or a sand pump.

(iv) Hydraulic Jet Method

This is the best and most efficient method for small diameter bores in soft soils. Water is pumped into the boring pipe fitted with a cutter at the bottom and escapes out through the annular space between the pipe and the bored hole. When the desired depth is reached, the pipes are withdrawn and the well tube with the strainer is lowered by the same process using a plug cutter with the plug removed instead of the ordinary steel cutter.

(v) Reverse Rotary Method

In this method, the water is pumped out of the bore through the pipe and fed back into the annular space between the bore and the central pipe. No casing is required in this method which is used only in clayey soils with little or no sand. This method is suitable for large diameter bores up to a depth of 150 m.

After the required depth is reached, the pipe with the cutter is taken out of the bore and the well pipe with the strainer is then lowered into the hole. The annular space between the bore and the well screen is then shrouded with pea gravel.

(vi) Sludger Method

In this method, the boring pipe with the cutter attached is raised and lowered by lever action and the bore filled with water from a sump nearby. This method is suitable for depths up to about 50 metres. This method is suitable for small diameter wells in soft soils and medium hard soils.

(vii) Casing of Bore Wells

Wells in soft soils must be cased throughout. When bored in rock, it is necessary to case the well at least through the soft upper strata to prevent caving. Casing is also desirable for the purpose of excluding surface water and it should extend well into the solid stratum below. Where artesian conditions exist and the water will eventually stand higher in the well than the adjacent groundwater, the casing must extend into and make a tight joint with the impervious stratum; otherwise, water will escape into the ground above.

(viii) Well Strainer and Gravel Pack

The openings in well strainers are constructed in such a fashion as to keep unwanted sand out of the well while admitting water with the least possible friction. In fine uniform strata, the openings must be small enough to prevent the entrance of the constituent grains. Where the aquifer consists of particles that vary widely in size, however, the capacity of the well is improved by using strainer openings through which the liner particles are pulled into the well, while the coarser ones are left behind with increased void space. A graded filter is thereby created around, with the aid of back-flushing operations or by high rates of pumping.

(ix) Yield Test for wells

The wells after their construction are tested for their yield, specific capacity, and aquifer parameters as per details given in **Annexure 4.3**.

(e) Artesian Water and Artesian Wells

Artesian aquifer is confined aquifer containing groundwater under positive pressure. Artesian aquifer has trapped water surrounded by layers of impervious rocks which apply positive pressure to water contained within aquifer. Artesian well is the name derived for a well from which water flows automatically under pressure and well is called "Auto-flowing" well which does not require a pump to yield water. An artesian well along with shallow and deep well are shown in Figure 4.11.

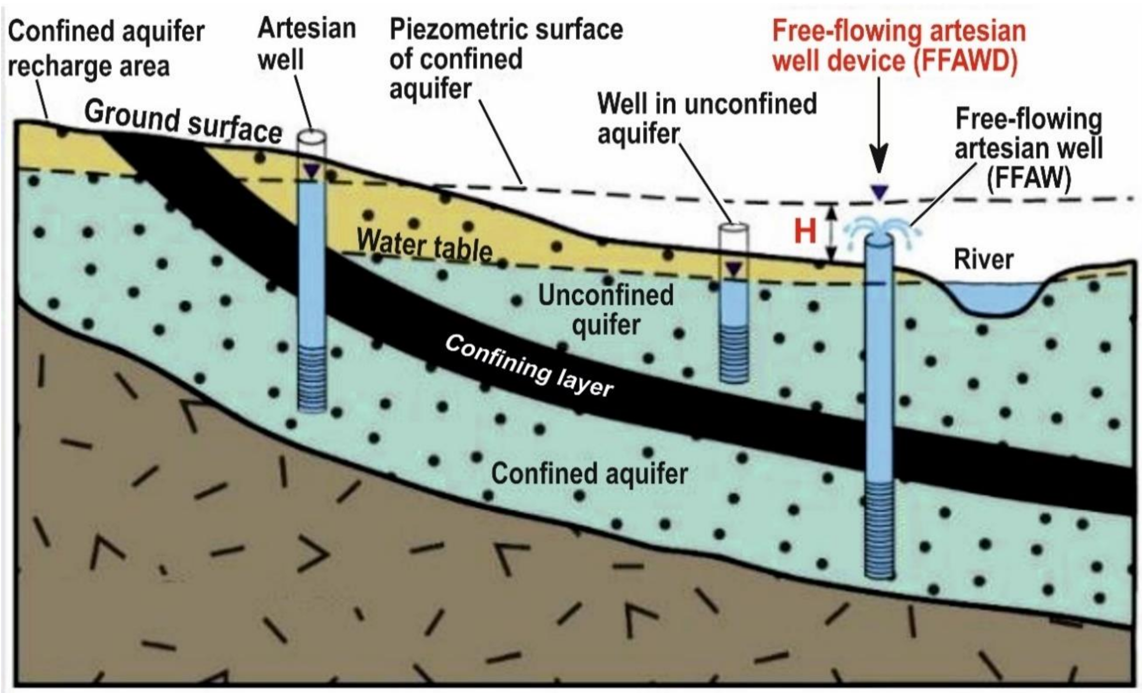


Figure 4.11: Artesian Well

Indian Artesian belt of great significance stretches along foothills of Himalayan Regions, commonly known as Bhabbar-Tarai belt, and are located in various states as follows:

(i) Artesian wells of Uttarakhand located in Tarai area. Udham Singh Nagar district is famous for auto-flow wells.
(ii) Tarai-belt of Jammu Province where spring-line exist at the contact of Bhubar and Tarai formation.
(iii) Artesian well water in the Malabar Coastal plain and Alleppy.
(iv) Artesian water in the Malabar coastal plain of southern Kerala.
(v) Artesian wells in the Great Rann of Kachchh, Gujarat.

Artesian wells work on the principle of Pascal's Law where a liquid at high pressure in one well will increase the height of liquid in another well.

4.10.3 Infiltration Galleries

(a) Wells vs. Galleries

These are horizontal drains made from open jointed or perforated pipes that are located below the groundwater table. Infiltration galleries offer an improvement over a system of wells. A gallery laid at an optimum depth in a shallow aquifer serves to extract the sub-soil flow along its entire length, with a comparatively lower head of depression. Moreover, in the case of a multiple system of infiltration wells, the frictional losses contributed by the several connecting pipes diminish the draw-down in the farther wells to that extent and the utility of a well becomes less and less in the total grid.

(b) General Layout

Essentially, a gallery is a porous barrel inserted within the permeable layer, either axially along or across the groundwater flow. A collecting well at the shore end of the gallery serves as the sump from where the infiltrated supply is pumped out. The collecting well is the point at which the maximum head of depression is imposed under pumping operation, the depression head being diffused throughout the length of the gallery to induce flow from the farthest reach.

The exact alignment of a gallery must be decided with reference to the actual texture of the sub-soil layers, after necessary prior investigations to map out the entire sub-soil. A gallery could be laid axially along a river or across a river. In both cases, the head of depression induced is the factor influencing the abstraction of the sub-surface flow into the gallery liner, and the zone of influence exerted along the entire length of the gallery line will have the same variations irrespective of the direction of the gallery.

(c) Structure of a Gallery

The normal cross-section of a gallery comprises loosely jointed or porous pipe or rows of pipes, enveloped by filter media of graded sizes, making up a total depth of about 2 ½ m and a width of 2 ½ m or above, depending on the number of pipes used for collection of the infiltrated water.

The gallery has necessarily to be located sufficiently below the lowest groundwater level in an aquifer, under optimum conditions of pumping during adverse seasons.

The galleries consist of either a single or double row of stoneware or concrete pipes loose jointed with cement lock filters. Perforated PVC pipes can also be used. The pipes are laid usually horizontally or to a gradient if aligned in the direction of flow. The coarse aggregate envelope in the pipe material is in three layers, followed by coarse and medium sand layers, as detailed below.

Filtering medium near pipeline – 18 mm broken stone.

2nd layer – 38 to 19 mm broken stone.

3rd layer – 12 to 6 mm broken stone.

4th layer – Coarse sand passing through a sieve of 3.35 mm size and retained on a sieve 1.70 mm size.

5th layer – Fine sand retained on 70-micron sieve and passing through 1.70 mm sieve.

The particle size distribution between each successive layer should preferably be based on a multiple of four. Precast perforated concrete barrels are also used as collecting pipes with the enveloping media on the three sides.

(d) Constructional Features

The constructional features during the execution of such galleries are of importance. Trenches are dug with adequate shoring or piling facilities right down to the required level decided upon for the invert of the gallery, which would normally be placed several metres below the sub-soil water level, a greater depth indicating a greater potential for the yield from the gallery. The gallery can be laid underwater, if dewatering the trench completely for the purpose is not feasible or economical. Manholes should be provided at intervals of about 75 m for inspection. These are sunk into the bed before the gallery is laid and the floor of these wells are taken a little below the invert level of the gallery pipe. The manholes are covered with RCC slab with watertight manhole frame and cover.

(e) Check dams

Under certain conditions, the provision of a sub-soil barrage or check dam across a river just downstream of a gallery system, helps in inundating the riverbed area over the gallery and providing permanent saturation of the sub-soil layers contributing to the yield through the gallery. The barrage is usually keyed into the riverbed on an impermeable layer and into the banks for it to function successfully. Incidentally, it would also save the gallery system against damages by scour during floods.

4.10.4 Radial Collector Wells

A well that has central caisson with horizontal perforated pipes existing radially into an aquifer is a Ranney well. It is also called a Radial collector well.

i. Constructional Details:

(a) De-sanding Operation while Driving Radials
An important operation in the driving of the drains is the operation of de-sanding of drain tubes of 200 mm to 300 mm diameter which will remain inside the sand bed being driven to a certain distance. An inner tube is then introduced into the drain which is used for sending a blast of compressed air for loosening and separating the fine particles of the alluvium at the head of the drain. When the compressed air is turned off, the pressure of the water, due to the head of the water table, enables the fine particles into the interior of the well to be carried until clear water without any fine particles is obtained.

(b) Suitability of Radial Collector Wells (RC-well) in Shallow Aquifers
Although boreholes are efficient method of groundwater extraction, but under special circumstances, collector wells are more suitable than dug well or borewell for groundwater extraction. This is where aquifer is thin, shallow and exhibits moderate permeability. Such conditions for example exist in Yamuna flood plain area in NCT Delhi. The large effective radius of shaft plus radials in a collector well make it a hydrogeological efficient method of maximising daily yields. Shallow alluvial collector wells can be constructed in such hydro-geological environment where shallow aquifer of high permeability exist such as the flood plain aquifer system of rivers.

An RC-well extracts groundwater with less drawdown at the well casing than what usually occurs at a traditional vertical well extracting water at same pumping rate.

(c) Features of a Radial Collector Well
These include:

- the horizontal perforated collector pipe which enables a large area of an aquifer to be exploited;
- the removal of fine sand and gravel in the path of the collector pipe, so that the artificial aquifer of much higher permeability is established;
- after construction, the collector pipe that serves as a sub-drain in a filter surrounded by a circle of coarse gravels of very large diameter.

ii. Design Details of a Radial Collector Well:

A collector well consists of a cylindrical well of reinforced concrete say 4 to 5 m in diameter, going into the aquifer to as great a depth of the sub-strata as possible, i.e., up to an impermeable stratum. Normally, the saturated aquifer should not be less than 7 m above the top of the radial pipes. From the bottom of the well, slotted steel pipes, normally of 200 mm to 300 mm diameter on the inside and going up to 30–35 metres in length are driven horizontally. The length is determined by the composition and yield from the aquifer. The drain tubes are made up of short length of pipes each 2.4 m in length which are welded to each other electrically one after the other.

These steel pipes are driven horizontally into the aquifer by means of suitable twin jacks placed in the well and crossing the steining of the well, through the special openings or portholes. At the same time, de-sanding operation is carried out through the head of the drainpipes. This operation is very important and results in the removal of all the fine particles in the alluvium thus increasing the draw-off. A radial well schematic diagram is placed at Figure 4.12.

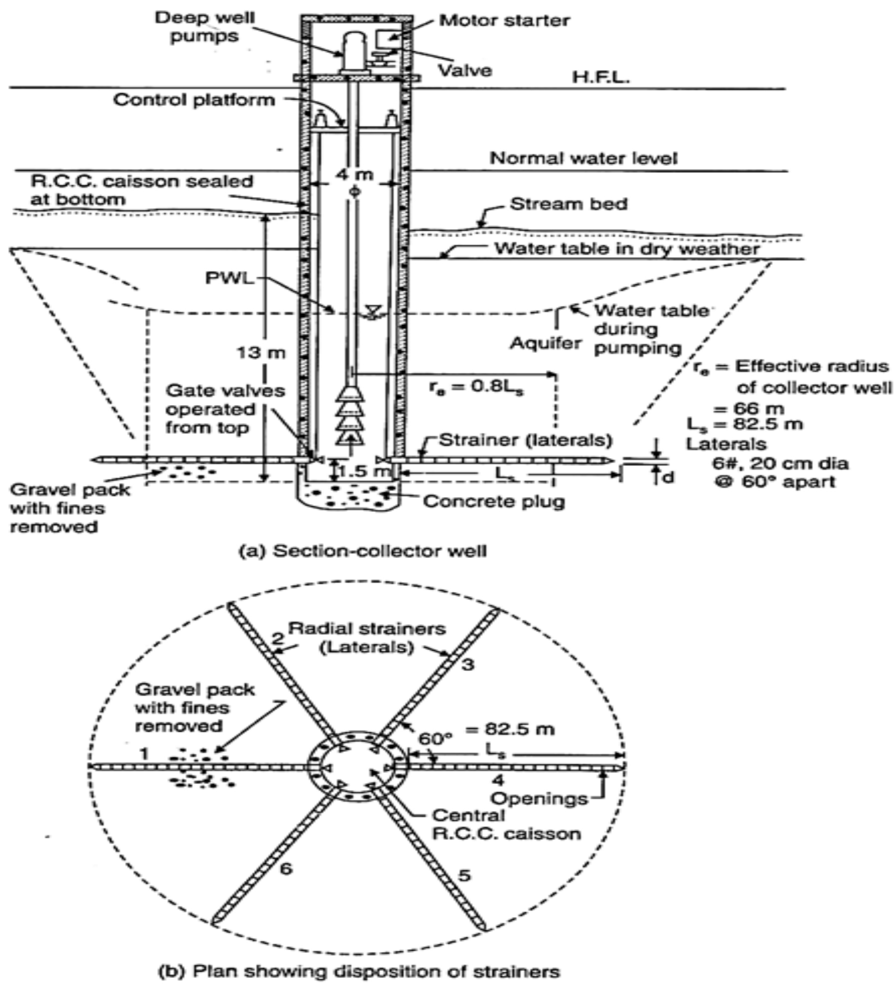


Figure 4.12: Radial Water Collector

(Source: https://in.pinterest.com/pin/323062973266073040/)

4.10.5 Filter Basins

When there is a perennial flow in a river and the sub-soil met with is hard rock below an average depth of 1.5 to 3 m, filter basins are constructed to take advantage of the perennial flow, assuming a filter rate similar to that of a slow sand filter. Sand in this area is removed and under-drains, usually loose-jointed stoneware pipes or perforated PVC pipes, are laid and covered with sand. The water from the under-drains will be led to a collecting well by CI or RCC pipes. The collecting well which is also used as pump house is located on the bank of the river.

4.10.6 Syphon Wells

When the depth of saturated aquifer is 20 - 30 m and the conventional wells and galleries cannot be laid to take full advantage of such depths, certain alternate devices have to be tried. A syphon well will be most suitable in this case. A syphon well consists of a masonry well, 4 - 5 m diameter, sunk to a shallow depth and sealed at the bottom.

4.10.7 Determination of the Specific Capacity of a Well

The specific capacity of a well is the discharge per metre of drawdown at the well. In the case of artesian wells, it is usually assumed that the specific capacity is constant within the working limits of the drawdown. The specific capacity decreases with duration of pumping, increase in drawdown and the life of well. High specific capacity can be ensured by proper selection of screens and gravel and thorough development.

(a) Measurement of Drawdown

The actual drawdown in wells under pumping is ascertained in several ways. In the case of shallow tube wells, dug or sunk wells, the more common method is to drop a weighted string up to the water level, before and during pumping and computing the difference. In the case of deep tube wells, a satisfactory procedure is to adopt the air pressure method.

The specific capacity may be determined either by the discharge method or by the recuperation method.

(b) Discharge Method

Using a pump discharging at a constant rate, the water level is lowered in a well and at intervals of time Δt, the water levels are noted.

The discharge equation for this method will be:

$$Q\Delta T = A\Delta h + Kh\Delta t. \qquad (4.2)$$

Where

Q = steady rate of pumping;
A = area of section of well;
K = specific capacity of the well;
h = average drawdown during the interval Δt
Δt = interval of time; and
Δh = depression during the interval Δt.

In the above equation, Q, A, and Δt are known, Δh is observed, h is measured, and K can be calculated for each set of observation.

The selection of the pump capacity should be such that a desirable depression is obtained finally. The time interval Δt should be such that the depressions during the time interval are neither too great nor too small.

When the water level is maintained constantly after a particular drawdown, the equation becomes:

$$Q\Delta t = Kh\Delta t \qquad (4.3)$$

Or

Q = Kh, i.e., the rate of pumping equals the yield for that particular drawdown and sp. cap. = Q/h

A practical way to confidently predict yields and drawdowns for larger dia. gravel packed permanent production wells is to construct two 65 mm dia. test-wells, 0.6 m apart, pumping one well with a centrifugal pump (about 30 KL/min. capacity) and measuring the drawdown in the other. The resulting discharge divided by the drawdown in the well 0.6 m away is the expected specific capacity of 1.2 m gravel packed well to be drilled at the site.

4.10.8 Maximum Safe Yield and Critical Yield

If the well is not developed to the full capacity of the aquifer, the maximum yield is limited by the maximum permissible drawdown at the well and by the size and the method of construction of the well. In the case of shallow tubular wells, the maximum permissible draw-down may be limited by the suction lift of the pumps or by the depth of the wells. In the case of masonry sunk wells as well as tube wells, the drawdown can be further restricted with a view to preventing sand blows which may disturb the aquifer unduly. Sand blows which help to remove the fines and help in the training of the yield are, however, desirable. The maximum quantity that can be drawn may be fixed with reference to the diameter of the well and the hydraulic subsidence value of the largest site of the particles proposed to be removed during the training of the yield to get the best results. This may be termed the critical yield.

4.10.9 Spacing of Wells

The amount of water which can be obtained from a system of wells depends upon the extent by which the water level can be lowered along the line of wells. The maximum amount of water obtained from a given system of wells would be when they are spaced enough apart so that their circles of influence will not overlap. If wells are deep and, therefore, expensive, they should be spaced to interfere comparatively to a lesser extent than the shallow wells which could be spaced closer. The extent of mutual interference can be judged by pumping tests on trial wells, or on those first sunk, the wells being operated at different rates and in various combinations.

4.10.10 Design of Water Well (Bored Well)

The main objectives of the bore well design is as follows:

1. The highest yield with minimum draw down consistent with aquifer capability
2. Good quality of water with proper protection from contamination
3. Water that remains sand free
4. Well should have long life (25 years or more)
5. Low initial cost

a. Well Structure

The well structure consists of two main elements – casing and intake zone.

b. Design Procedure

Selecting the casing diameter and material

Casing diameter of the well is important because it will significantly affect the cost of the structure. Therefore, following considerations should be given while selecting the casing pipe:

1. The casing must be large enough to accommodate with enough clearance for installation of pump, passage of drilling tools and development equipment.
2. The diameter of casing must be sufficient to assure that the up-hole velocity is 1.5 m/sec or less.
3. The casing diameter should be kept 50 mm larger than the pump bowls.

4. The casing should have smooth exterior to minimise resistance against the formation due to friction.
5. The casing should have sufficient wall thickness to resist the stresses from the placement and subsequent production.
6. The casing must be capable to withstand the corrosive groundwater.
7. In deep wells that have both static and high pumping water levels, the casing diameter can be reduced at a depth below the lowest pump setting to reduce material cost.

The following Table 4.11 gives the recommended diameter of well casing for various pumping rates:

Table 4.11: Recommended Diameter for Well Casing

S. No.	Expected well yield L/min	Internal Diameter of well casing (cm)		Nominal size of pump bowel (cm)
		Minimum	Maximum	
1	400	12.5	15	10
2	400 – 600	15	20	12.5
3	600 – 1,400	20	25	15
4	1,400 – 2,200	25	30	20
5	2,200 – 3,000	30	35	25
6	3,000 – 4,500	35	40	30
7	4,500 – 6,000	40	50	35
8	6,000 – 10,000	50	60	40

Source: Ragunath, 2007

The Table 4.12 shows the Recommended Minimum Diameter for Well Casings and Screen.

Table 4.12: Recommended Minimum Diameter for Well Casings and Screen

S. No.	Well Yield (m^3)/day	Normal Pump Chamber Casing Diameter (cm)	Surface Casing Diameter (cm)		Normal Screen Diameter(cm)
			Naturally Developed Wells	Gravel Placed Wells	
1	<270	15	25	45	5
2	27 – 680	20	30	50	10
3	680 – 1,900	25	35	55	15
4	1,900 – 4,400	30	40	60	20
5	4,400 – 7,600	95	45	65	25
6	7,600 – 14,000	40	50	70	30
7	14,000 – 19,000	50	60	80	35
8	19,000 – 27,000	60	70	90	40

(Source: US Bureau of Reclamation, 1977)

i. Design for sanitary protection

Well cap and well seal are both designed to cover the top of a water well. Sanitary well caps and grout seal are primarily installed to especially safeguard against the bacterial contamination. All drinking water wells supplying potable water should be provided with continuous sanitary protection. Contaminated from surface drainage or low quality of water can move downward through the annulus between the casing and bore hole wall. The annulus around the casing must be sealed either by placing a cement grout in the annulus. Sometimes bentonite is used in place of cement.

ii. **Disinfection of Wells and Pipelines**

New wells as well as those after repairs have to be disinfected by heavy dose of chlorine. The doses applied are generally of the order of 40 to 50 mg/L of available chlorine and bleaching powder is usually employed. For pipelines, when a section of water main is laid or repaired, it is impossible to avoid contamination of the inner surface, therefore, disinfection is needed. The further details about disinfection of wells and pipelines are provided in **Annexure 4.4**.

4.11 Ground Water Monitoring

The current groundwater monitoring focuses on collecting data at a broader scale, but there is a need to decentralize these efforts and expand the data collection to numerous monitoring points. Inclusion of private wells including borewells is apparently needed to avoid capital intensive drilling of new wells. However, there are challenges in field in terms of accessing borewells for measurements and constraints posed by available sensor-based technologies in pumped wells. Based on the principle, "what needs to be managed, needs to be measured", it is imperative to measure the groundwater resource and manage it efficiently.

Non-contact Acoustic Technology to monitor ground water

There are non-contact acoustic technologies available as an alternate to the current sensor-based technologies which are capital and maintenance intensive, invasive, time consuming, short working life cycle and difficult to use. These non-contact acoustic technologies are available as mobile applications and as IoT devices and those are simple, handy, cost effective and scalable enabling quick data collection across large geographies. There is no need to open the borewell assembly and thereby quick measurements possible. Applications of such technologies are wider across a range of ground water monitoring programmes and other programmes where ground water data collection is involved.

These technologies can be used in Urban Aquifer Management plan under AMRUT 2.0 and Atal Bhujal Yojana. The Ministry of Housing and Urban Affairs (MoHUA), Government of India has encouraged start-up companies. Some of such start-up companies have successfully used an aquifer management plan for funding under AMRUT 2.0 project.

Expected capabilities:

- Simple and easy to use mobile app
- May not require any sensor or equipment.
- Measures water levels within few minute
- No need to open the borewell.
- Works on borewells with pump assembly
- Geo Location and Geo-Fencing facility

Expected Outcome:

- Help assess the impact on water availability due to pumping and recharge
- Helps delay the early drying of borewells and sustain it for longer durations
- Predict water availability
- Helps save borewell clogging and pump repairs caused by the dry operation of pumps
- Helps save electricity due to the regulated consumption of borewell
- Helps adapt improved water planning based on known water availability

These technologies have abilities to transfer data to servers and dashboards enabling real time data monitoring and offer immense opportunities for predictive analysis for developing assessments and advisories for various stakeholders. These technologies can be accessed by individuals like farmers, municipalities and urban households and can empower them to monitor their own sources and manage them efficiently thus helping to decentralize the ground water management further.

4.12 Groundwater Recharging Methodologies

4.12.1 Conventional Recharging Methods

Groundwater recharging methods are broadly classified into four categories of techniques. These are as given Table 4.13 below:

Table 4.13: Groundwater Recharging Techniques

(i)	**Direct surface techniques** • Flooding • Percolation ponds/basins • Ditch and Furrow system • Over-irrigation
(ii)	**Direct sub-surface techniques** • Injection wells • Recharge pits/shafts • Dug well recharge • Bore-hole flooding • Cavity fillings
(iii)	**Combined surface and sub-surface techniques** • Basin or percolation tanks with pit-shaft or bore-wells.
(iv)	**Indirect techniques** • Induced recharge from surface water sources • Aquifer modification

In addition to above, the following groundwater conservation structures also help arresting of sub-surface flows:

(i) Groundwater dams or sub-surface dykes
(ii) Hydro-fracturing and blasting in hard rock areas
(iii) Cement sealing of fractures through specially constructed bore wells to consuming sub-surface flow and augmenting bore-well yield.

4.12.2 Managed Aquifer Recharge (MAR) Innovations

Adoption of innovative MAR approaches to pursing sustainable water management is an inescapable necessity of time particularly when changing climates are impacting water and water infrastructure system. Achieving sustainable and secured urban water supply and services would need to use holistic integrated urban water management (IUWM) framework. The suggestive MAR innovations include aquifer storage and recharging system (ASR), river and lake bank filtration system (RBF/LBF) with storage goal and potable water use. Also, MAR system such as modular rain tank system, in-stream modifications and recharge, conventional RWH system with non-storage goals can be used to support non-potable urban water supply uses. Role of stake holders and water managers is imperative to sustainable urban water management.

1. **Urban Aquifers & MAR Systems:** The depth, limit, and extent as well as empty-storage space of both alluvial and hard rock aquifers are pre-requisite to planning MAR systems. Aquifer sensitivity maps, maps of potential contaminant sources and maps showing direction and rate of movement of groundwater flow are of paramount importance to develop aquifer recharging plans. There has to be compatibility between source recharge water and native groundwater under recharge. Due to complex nature of aquifer systems, the complexity of hydrogeologic-framework is also required to be investigated in detail prior to recommending the design, suitability and feasibility of MAR methods and recharging structures.
2. **Source Water for Recharging:** Managed recharge to aquifers can be used to store water from various sources such as urban storm water from roofs of houses and buildings, pavements and roads which shed water from their embankments. Source water also includes water from rivers and lakes, ponds, treated wastewater and desalinised seawater. Recycled urban storm water can be stored in aquifer underlying parks &gardens, sports complexes and flyovers for non-potable uses.

   Urban water systems are faced with impacts of climate change, rapid urban population growth, population migration from rural to urban centres as well as deteriorating age-old water infrastructure. The need to manage urban water supply has therefore been an urgent necessity and inescapable necessity of time. The integrated urban water management (IUWM) seeks to integrate planning, management and, community participation to building climate –resilient city and township water supply and sanitation system. IUWM is holistic management of urban water supply, sanitation, storm water and wastewater to yielding sustainable socio-economic and environmental objectives. Various IUWM application tools can help water utilities manage the threat and menace of climate change.
3. **Priority MAR Methods:** The Empty storage capacity of urban Aquifers classify themselves into priority category areas (viz., Priority I and Priority II Category Area) to the recharging of groundwater.

   Priority I category MAR project will involve high value use areas as potable supply water and Priority II as lesser value use areas for non-potable water use such as for horticulture and watering of parks and gardens. It is imperative to list out the Priority I and II MAR Projects. These are given in Table 4.14 below.

Table 4.14: MAR Priority Projects

i	Priority I MAR Project (For potable water use)	Recharge System • Aquifer storage and recovery (ASR) and aquifer Storage, Transfer and Recovery (ASTR) Well System. • Riverbank Filtration (RBF) and Lake Basin Filtration (LBF) System
ii	Priority II MAR Projects (For Non-potable water use)	• Check dams, Gabion and Nala bunds. • City Roads, Sports complex, Fly overs • Shafts and Trench driver bore wells. • Pond basins.

4.13 Integrated Water Resources Management (IWRM):

The challenges confronting today's major cities are daunting, with water management standing out as one of the most serious concerns. Access to potable water from pure sources is scarce, necessitating the treatment of alternative water sources at a high cost, while the volume of wastewater continues to rise. City dwellers in many areas of the Country lack good quality water and fall ill due to waterborne illnesses. As cities seek new sources of water from upstream and discharge their effluent downstream, surrounding residents bear the adverse effects. The hydrologic cycle and aquatic systems, including vital ecosystem services, are disrupted.

Water is a primary requirement for survival, yet its effective management in terms of diversion, transport, storage, and recycling is one of the most elusive targets. An efficient water supply, sanitation, and allied services have tremendous socio-economic and health benefits, a fact that has been reiterated by United Nations' Sustainable Development Goal (U.N. SDG) number 6.

The level of water availability, quality of piped water and the treatment, reuse, and recycling of used water are frequently regarded as proxy indicators reflecting the level of development of a nation. Government schemes to provide clean, safe water, and necessary sanitation facilities to every citizen in India, have served to reinforce our national commitment for better water services. However, a great deal of preparedness is necessary from the grassroots level to enable superior water resource management. India has a two-tier governance system for management of its water resources – the first tier consists of the Central Governmental agencies which deal with policy matters on inter-state rivers, flood management and international water issues, while the second level consists of State water/water resource authorities/ULBs, which are responsible for management of water resources, water supply and sanitation services in the respective states.

Water is usually pumped to large distances and high elevations, greatly increasing the associated energy costs. While surface water is the primary source in most locations, there is significant dependence on groundwater in regions where surface water sources do not provide reliable supply across the year. In the absence of appropriate recharge measures results in depleting groundwater resources, which in turn leads to saltwater intrusion in coastal aquifers, and other problems associated with deterioration of groundwater quality.

Ever-growing urban populations have intensely stressed available water resources for any city or town. Water demand is one of the major uncertainties for operation and management of a water distribution system (WDS), which varies seasonally and regionally.

The per capita availability of water in India is less than 1,000 m^3/capita/year based on the estimated utilisable water resources of 1,123 BCM (Ministry of Water Resources (MoWR), 2012), about 1,588 m^3/capita/year (Office of the Registrar General India, 2011) which makes us among one of the most water stressed countries in the world. The population in India has increased by about 181.5 Million from 2001 to 2011(Office of the Registrar General India, 2011), and the similar rate of increase is expected in the near future as well. With this rate of population increase, stress on water resources is inevitable. It is suggested that by 2030, India will face water scarcity amounting to 50% of its water demand, or 75 BCM (billion cubic metres) (United Nations' Children Fund (UNICEF), 2013).

The National Water Policy (Ministry of Water Resources (MoWR), 2012) recommends priority of water allocation to be retained for drinking and sanitation followed by agriculture and supporting livelihood for the poor. The policy emphasises on avoiding wastage on unnecessary uses and utilising water judiciously.

Providing adequate quantity and safe quality of drinking water are key priorities of most Indian States, and there are numerous challenges that inhibit accomplishing such objectives. Water quantity estimations are performed by assessing supply and demand levels. Supply-side management involves infrastructure optimisation, preventive maintenance, minimisation of losses, metering of connections, etc. Demand management, on the other hand, involves social awareness, effective usage of supplied water, pricing, billing, and minimisation of losses. Parity between demand and supply levels is necessary for efficient distribution and reducing residence time of water within the

WDS, which is required to preserve the integrity of the water quality. Water pricing and household metering has been seen to reduce additional demands.

Even urban water demand increases due to growing populations, water supplies may become scarce as precipitation patterns, river flows, and groundwater tables change (UN-Habitat, 2011). Some sources may become unsuitable for certain uses (e.g., salinity may limit water for agricultural use), and the cost of water treatment may rise (e.g., eutrophication may require additional treatment of domestic water). For some fast-growing desert and semi-desert megacities, water scarcity may be severe. Climate change is likely to affect water supply technologies, primarily through flood damage, increasing treatment requirements and reducing availability and operational capacity. Extended dry periods will increase the vulnerability of shallow groundwater systems, roof rainwater harvesting, and surface waters.

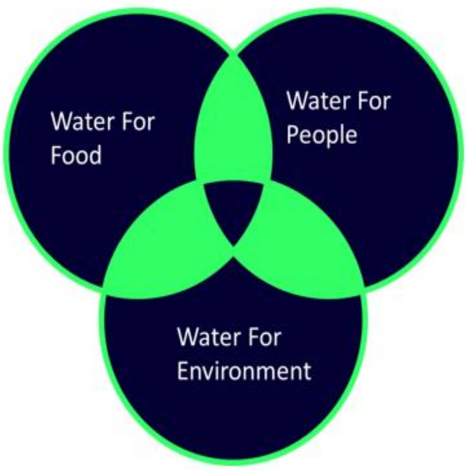


Figure 4.13: Definition Sketch of IWRM
(Source: IWRM, 2005)

Climate change also poses significant threats to the reliability and resilience of our water sources. Clearly, sustainable water resources management calls for an integrated approach and constant monitoring and re-adjustment of all its components.

It is prudent to note that the Integrated Urban Water Resource Management (IUWRM) is a subset to IWRM which is more aligned towards water management on broader and larger catchment scale, on broader principles as mentioned in Figure 4.13. IUWRM is more aligned towards managing the water resources on a sustainable basis in an urban setting. In the following discussions, IWRM is used more to represent IUWRM.

4.13.1 Rationale of IWRM

In the past, water supply, sanitation, used water treatment, stormwater drainage, and solid waste management have been planned and delivered largely as isolated services. Conventional Urban Water Management seeks to ensure access to water and sanitation infrastructure and services. Conventional urban water management strategies, however, have strained to meet demand for drinking water, sanitation, used water treatment, and other water-related services. Some cities already face acute water shortages and deteriorating water quality.

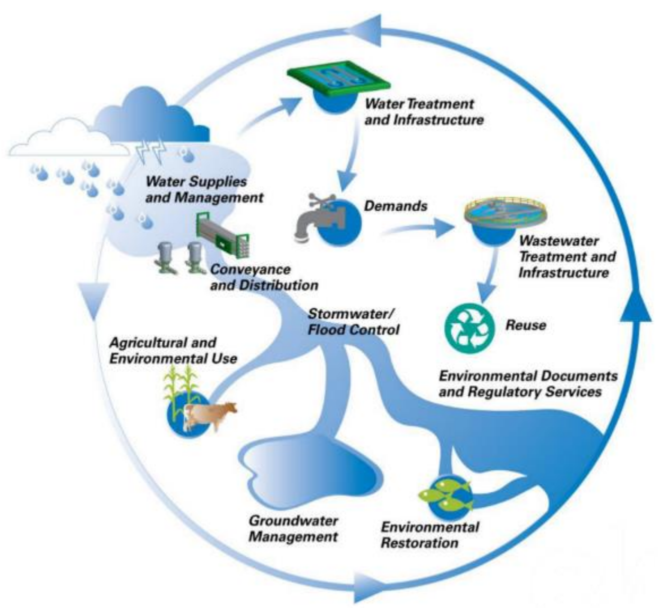


Figure 4.14: The Principles of IWRM
(Source: www.google.co.in/natural + resources)

It must also manage rainwater, used water, storm water drainage, and runoff pollution, while controlling waterborne diseases and epidemics, mitigating floods, droughts, and landslides, and preventing resource degradation. Even though conventional urban water-management strategies have been unable to respond to existing demands, more will be asked of urban water management in the future. Given the challenges posed by urban growth and climate change, conventional urban water-management practice appears outdated.

A range of authorities, each guided by distinct policies and pieces of legislation, continue to oversee water subsectors at the city level. The traditional urban water-management model has failed to distinguish between different water qualities and identify uses for them. As a result, high-quality water has been diverted to indiscriminate urban water needs (Van der Steen, 2006). This issue is not confined to city boundaries: basin-level management often neglects to acknowledge the cross-scale interdependencies in freshwater, used water, flood control, and storm water. Water is extracted from upstream sources and delivered to urban areas, where it is used and polluted, then re-channelled – often untreated – downstream.

Water issues often remain disconnected from broader urban planning processes. This problem is particularly evident in developing countries, where modern urban development, associated with the design of physical human settlements and land-use zoning schemes, still hold sway (UN-Habitat, 2009).

IWRM includes assessments to determine the quantity and quality of a water resource, estimate current and future demands, and anticipate the effects of climate change. It recognises the importance of water-use efficiency and economic efficiency, without which water operations cannot be sustainable. It also recognises that different kinds of water can be used for different purposes: freshwater sources (surface water, groundwater, rainwater) and desalinated water may supply domestic use, for example, and used water (black and grey water) can be treated appropriately to satisfy the demands of agriculture, industry and the environment as explained in Figure 4.14. With efficient new desalination technologies, saltwater has become an accessible water source.

Therefore, integrated urban water resource management (IWRM) promises a better approach than the current system, in which water supply, sanitation, storm water and used water are managed by isolated entities, and all four are separated from land-use planning and economic development. IUWRM calls for the alignment of urban development and basin management to achieve sustainable economic, social, and environmental goals.

The traditional fragmented sectoral approach and that of the cross sectoral integrated approach are respectively shown in Figure 4.15 and Figure 4.16.

Demerit of the traditional fragmented sectoral approach is that it can create problems and pushing the system to unsustainable use and poor services. For example, City administration makes drinking water reservation in the live storage of the dam. But sometimes, if dam authority releases excess water for irrigation, then there will be chaotic conditions, lot of tankers will have to be used to accommodate domestic use. Moreover, if dead water from dam is utilised it will pose problems of taste, colour and odour and there will be unrest in city customers. Similarly, putting domestic sewage or releasing industrial pollutions will pose health problems. In IWRM since there is cross-sectoral integrated approach, such situations are avoided and system runs efficiently. Thus, IWRM is a process which helps to deal with water issues in a cost-effective and sustainable way.

4.13.2 Objectives and principles of IWRM

Objective of an IWRM Plan is to promote development that co-ordinates management of water, land and related resources so as to maximise the resultant economic and social welfare. One of the major

Figure 4.15: Traditional Fragmented Sectoral Approach

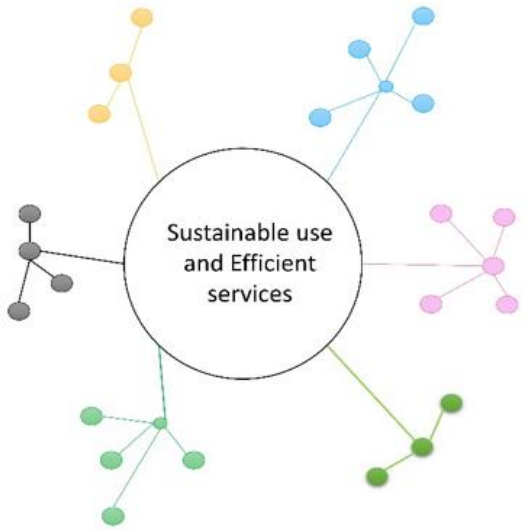


Figure 4.16: Cross-Sectoral Integrated Approach

aspects considered in development of this approach includes identification and development of an optimised solution that is based on techno-economic evaluation of various available water sources or combinations thereof.

The goals of urban water resource management are to ensure access to water and sanitation infrastructure and services; manage rainwater, used water, storm water drainage, and runoff pollution; control waterborne diseases and epidemics; and reduce the risk of water-related hazards, including floods, droughts, and landslides. All the while, water management practices must prevent resource degradation.

Under IWRM Plan, Triple-Bottom-Line (TBL) Principles has been used to help identify the most preferred water infrastructure solution.

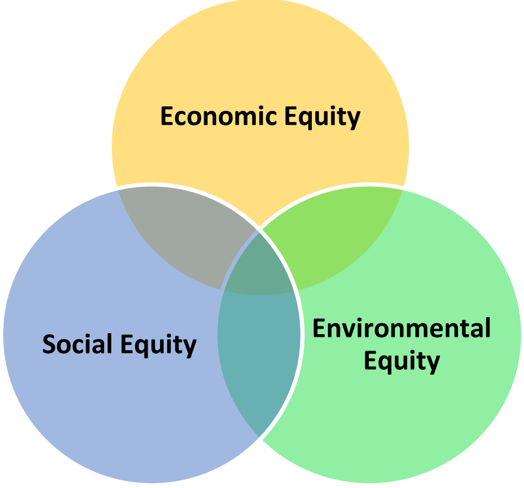


Figure 4.17: The Triple Bottom Line Principle

The most preferred solution is to be an optimal mix of social, environmental and economic benefits as well as being practical and recognises that ideal and perfect solutions seldom exist in the real-world as shown in Figure 4.17.

Integrated urban water management (IWRM) offers a set of principles that underpin better co-ordinated, responsive, and sustainable resource management practice. It is an approach that integrates water sources, water use sectors, water services, and water management scales:

- It recognises alternative water sources.
- It differentiates the qualities and potential uses of water sources.
- It views water storage, distribution, treatment, recycling, and disposal as part of the same resource management cycle.
- It seeks to protect, conserve and exploit water at its source.
- It accounts for nonurban users that are dependent on the same water source.
- It aligns formal institutions (organisations, legislation, and policies) and informal practices (norms and conventions) that govern water in and for cities.
- It recognises the relationships among water resources, land use, and energy.
- It simultaneously pursues economic efficiency, social equity, and environmental sustainability.
- It encourages participation by all stakeholders.

Under IWRM, supply management and demand management are complementary elements of a single process. There is no one-size-fits-all model nor is any single method sufficient. Rather, the mix of approaches reflects local socio-cultural and economic conditions.

4.13.3 Development of IWRM Plan

Global Water Partnership (GWP) as part of the Dublin-Rio statement of 1992 defines “Integrated water resources management is based on the equitable and efficient management and sustainable use of water and recognises that water is an integral part of the ecosystem, a natural resource, and a social and economic good, whose quantity and quality determine the nature of its utilisation”. An IWRM Plan adopts sustainable, resilient and cyclical water resources utilisation in an urban setting. In essence, it reflects the ‘Whole to Part’ approach in managing water on a city or urban centre level, where the water demand emanates from multiple users, and water supply comprises of different sources from single or multiple watersheds. Efficient and equitable distribution of water; collection, treatment, and safe disposal and/or reutilisation of used water; creating financial sustainability and concerted stakeholder engagement forms the core of an IWRM Plan.

4.13.4 Vision and Scope of IWRM Plan

IWRM is the only feasible way forward to ensure water security for Indian cities. This integrated approach requires collaboration with multiple stakeholders from diverse backgrounds- ranging from hydrology, hydraulics, chemists, microbiologists, management, data sciences to social sciences among others. Some of the key considerations while building IWRM systems are summarised as below:

- IWRM solutions should be uniquely tailored for each catchment and city. One must remember that IWRM solutions are not a one-size-fits-all but should be customised for the local hydrology, climate, geology, water use patterns, demographics and other relevant factors. For example, separate IWRM plans should be developed for Mega, Tier I and Tier II cities, to effectively reflect local conditions, treatment capabilities and environmental requirements.
- Multiple sources of water should be delineated by cities, which have satisfactory levels of quality and reliability, now and in the future.
- These sources must be protected from external contamination to avoid excessive treatment costs at subsequent stages.

- WDSs should be carefully planned, with extensive monitoring to improve control over the quantity and quality of water at various stages from catchment-to-consumer. Data-driven analyses should be used to model system behaviour and predict future performance for various scenarios of uncertainty.
- Used water should be viewed as a resource and recycled into the system to augment water availability. Cities should aim for innovative uses of secondary or tertiary treated water in order to minimise the burden on freshwater resources.
- Water balance studies need to be conducted at a city-scale, to account for all sources, demands and recovery channels.
- Tertiary treated water can be used to create natural river systems, groundwater recharge systems, or can be additionally treated and blended with freshwater resources to make it fit for drinking and other purposes.
- Any IWRM project should account for various scenarios of urbanisation, population growth and climate change, and be prepared with suitable responses.

Development of an Integrated Urban Water Resources Management Plan requires a multi-disciplinary, holistic, and systematic approach. It should promote practices that are focused towards delivering solutions that will create a desirable future for people, business, and the environment in the project area, and forms the basis for developing a healthy state of dynamic balance between human, natural, and economic and environmental/ecological systems.

4.13.5 Approach

The overall project approach for delivering a sustainable IWRM Plan is built on three fundamental objectives as follows:

- To develop an optimised solution, based on techno-economic evaluation of alternatives to effectively utilise all the available water resources in a sustainable manner to address the water demands as development grows in the future.
- To develop a robust suitable operating model and large data management tools that are highly efficient in optimising the operations of water infrastructure in an effective and accurate manner and will also act as a dynamic decision support tool for managing magnitude/multitude of scenarios.
- To develop a sustainable Integrated Urban Water Resources Management Plan that incorporates a strategic prioritisation of planned projects.
- To achieve the above-mentioned objectives, the project activities are distributed in three consecutive stages as shown in Figure 4.18**.**

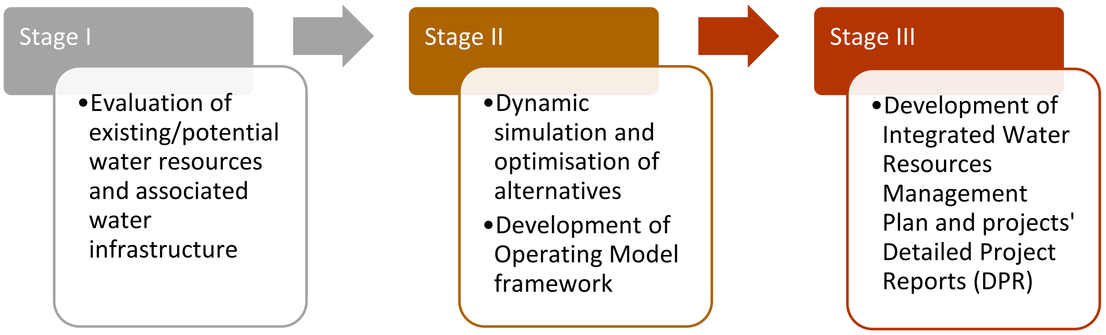


Figure 4.18: Staging of Project Activities

The guidance for developing an IWRM following the stages presented in Figure 4.18 are briefly explained below.

4.13.6 Stage I – Evaluation of Existing Water Resources and Infrastructure

4.13.6.1 Overview of Existing Resources

A holistic water source identification should be performed to develop a diverse water portfolio. Identification of sources can be done based on multi-criteria analyses, where selection of each source is assessed based on its societal, environmental, economic, and technical impact. It is important that all available water sources, such as surface water, ground water, harnessed rainwater, used water, recycled water, inter basin water transfer, seawater, non-revenue water (NRW), etc. are identified and evaluated before finalising the water portfolio.

While the available quantity and quality of the source water is of primary importance, the reliability of the source should be keenly analysed too. Reliability refers to the dependability of the source to provide the requisite quantity and quality of water across various seasons in a year, and also several years down the line as urban water demands grow. Ideally, an urban settlement should have not just one, but multiple reliable sources (both surface water and groundwater) to provide water under various scenarios of climate change, land use changes and/or exigencies like droughts. While calculating the water demands of an urban area, care should be taken to make allocations for recreation, environment and ecology, and urban river rejuvenation besides the usual water demands for human and economic development.

Perform quantitative and qualitative assessment for all potential sources of water in the form of ‘Strengths Weaknesses Opportunities Threats’ (SWOT) Analysis. The SWOT analysis will form the basis for identifying the aspects that needs to be addressed prior to development of an IWRM Plan.

The details of various potential sources are as under.

- Surface water
- Groundwater
    - Groundwater depths
    - Hydro-Geochemistry/Groundwater quality
- Treated used water and recycled water
- Rainwater (rooftop and/or at catchment level)
- Storm water
- Seawater desalination
- Water demand management
    - NRW reduction

a. Surface Water Resources

The initial step towards ensuring safe drinking water supply is protecting the surface water source from contamination via (untreated) domestic, agricultural, industrial sewage. According to the World Water Assessment Programme (WWAP), 70% of the untreated domestic and industrial waste is dumped into water bodies, which renders the source unusable or leads to very high treatment costs. Increasing pollution and rapid depletion of surface water sources often increases the dependence on groundwater sources for supply.

Availability of the reliable source through the project period is the most important parameter that needs to be considered. Allocation of surface waters to drinking and other purposes from water resources department of the state, which handles all water requirements is the key factor.

Surface water sources may contain high levels of faecal coliform, indicating the necessity for thorough disinfection. Treatment technologies should be ascertained not only based on current pollutant profiles, but also by anticipating the occurrence of emerging contaminants such as pharmaceutical and personal care products (PPPs), pesticides and endocrine disrupting compounds (EDCs), and micro-plastics.

b. Ground Water Resources

These groundwater sources have been seen to contain high levels of nitrate, arsenic, and fluoride. Wells, which are commonly used for extraction of groundwater resources, are vulnerable to contamination from surrounding areas, as are the surrounding aquifers if polluted waters are allowed to seep into the ground. Indiscriminate groundwater withdrawal in coastal regions exacerbates saltwater intrusion, further deteriorating the quality of groundwater resources in the area. Clearly, the need for robust source protection measures cannot be undermined. Investments in source protection translate directly into savings in treatment costs and source replacement costs. Further, it should be noted that the suitability of a source to provide drinking water to a community should be ascertained not only on the basis of its yield/water availability, but also on its quality, which directly impacts public health.

For groundwater development status, below Table 4.15 presents the guideline that can be adopted. Groundwater status mapped by CGWB/State Ground Water Department could be the starting point for such analysis. Other survey data obtained by educational, research or private organisations can also be used.

Table 4.15: Groundwater Balance

Groundwater Resource Balance		
Annual replenishable groundwater resource	Monsoon season	Recharge from rainfall
		Recharge from other source
	Non-monsoon season	Recharge from rainfall
		Recharge from other source
	Total	
Extraction during non-monsoon season (loss)		
Net annual groundwater availability		
Annual groundwater draft (demand)		Urban irrigation
		Domestic and industrial users
		Total
Groundwater deficit volume		
Stage of Groundwater Development (%)		

c. Treated used water and reuse water

Typically, the treated used water is either directly or indirectly discharged into the river or disposed on open land. Only a small portion of the treated used water that is currently being generated by various Used Water Treatment Plants (UWTP) is used for non-potable needs. Need-based treatment for the consumer refers to a level of treatment required to satisfy water quality for intended use. For example, if water is intended to be used for flushing or gardening, it does not have to be treated to drinking water standards. This approach not only ensures a net saving in treatment costs, but also reduces the burden on high-quality drinking water.

Recycling and reuse close the loop between water supply and used water disposal. Integration of these two water management functions requires forward-looking planning, a supportive institutional setting, co-ordination of infrastructure and facilities, public health protection, used water treatment technology and siting appropriate to end uses, treatment process reliability, water utility management, and public acceptance and participation.

As many cities are now focusing on circular utilisation of treated used water and recycled water, depending on the end uses (indirect potable or direct non-potable), water quality plays an extremely important role. Such an approach is also linked with Need Based Treatment, wherein the product water is treated at different level based on end user requirement (irrigation, land-based disposal, disposal to water body, industrial uses)

d. Rainwater

Rainwater harvesting can help address water scarcity at the household level and may be easy and cost-effective to implement. Rain water harvesting provides a direct water supply and can recharge groundwater, while reducing flooding. Such measures may be an immediate solution to accompany long-term infrastructure improvements in water supply and drainage.

e. Storm water

Storm water can mitigate intense rainfall events and enhance local water sources. Cities that suffer from flooding have several options for urban storm water management, such as using retention ponds, permeable areas, infiltration trenches and natural systems to slow the water down.

f. Seawater Desalination

Desalination systems could be adopted to supplement water availability in coastal areas, to reduce the stress on freshwater resources. In cities that have exhausted most of their renewable water resources, desalinated water meets both potable and industrial demand. The cost of producing desalinated water was estimated about Rs. 48.80 per cubic metre (levelled tariff) which is being paid by Govt. of Tamil Nadu and Chennai Metro Water Supply and Sewerage Board to a Private company for a period of 25 years starting from the year 2009–10 (100 MLD desalination plant set up at Minjur, Chennai on DBOOT Basis).

g. Non-Revenue Water

Non-revenue water (NRW) is an issue with almost all water supply utilities in India. It includes physical and commercial losses and free authorised water for which payment is not collected. The average NRW in India is about 38%, just above the global average range of 30% to 35% reported by the World Bank. The control of NRW will conserve the freshwater resources and prevent the augmentation of water resources and postpone the investment.

4.13.6.2 Source Water Quality

Extensive catchment-to-catchment-via-consumer (C2C via C) monitoring should be carried out by water supply authorities/boards. The concept of C2C via C refers to monitoring of water at every step of its transmission: from the source to consumer (as drinking water) and subsequently from consumer back to a source (as treated, partially treated/untreated used water). C2C via C monitoring enables water boards to keep track of the quantity and quality of water being generated, used, re-used, and eventually sent back to the catchment. Reliable flow, pressure, and water quality sensors, placed at optimal locations along the water supply system can be beneficial to detect both pressure and discharge/flow rates, leaks, water quality anomalies, contamination events, etc. Also, historic databases generated, can be used for better system monitoring, control, and behaviour prediction. Internet of Things (IoT)-based sensor measurements should be carefully validated and curated, and their accuracy should be cross-checked through regular calibration before further analysis. This should be a continuous process.

While the chief goal of water distribution in a city is to achieve a per capita target of supplied water, it is equally important to prioritise water quality during planning. The quality of water supplied has a direct bearing on human health and well-being. Improved quality of supplied water will not only reduce occurrences of various water-borne diseases, but also reduce the dependence on home-treatment units (such as RO units) or bottled water. Ensuring the supplied water meets the recommended standards, both spatially (across all locations in a city, state, or country), diurnally, and temporally (all seasons of a year, both during monsoon and low-flow periods in a river) is a crucial step towards IWRM. Some important water quality aspects have been discussed below. This should not be treated as an exhaustive list, but rather an indicator for priority areas to be explored.

I. Source and Well-Head Protection

Wells, which are commonly used for extraction of groundwater resources, are also vulnerable to contamination from surrounding areas, as are the surrounding aquifers if polluted waters are allowed to seep into the ground. These groundwater sources have been seen to contain high levels of nitrate, arsenic, and fluoride. Indiscriminate groundwater withdrawal in coastal regions exacerbates saltwater intrusion, further deteriorating the quality of groundwater resources in the area. Clearly, the need for robust source protection measures cannot be undermined. Investments in source protection translate directly into savings in treatment costs and source replacement costs. Further, it should be noted that the suitability of a source to provide drinking water to a community should be ascertained not only on the basis of its yield/water availability but also on its quality, which directly impacts public health.

II. Need-based Water Treatment

As the water quality from surface and groundwater sources varies considerably, need-based water treatment processes should be adopted for removal of pollutants. Need-based treatment for the supplier refers to targeting the commonly occurring pollutant groups in particular source of water. This kind of treatment relies on prior knowledge of the common pollutants found in a source. For example, groundwater sources are found to contain higher levels of arsenic or fluoride. Additional arsenic/fluoride treatment units must be installed along with the conventional treatment system, keeping in mind that the waste sludge thus generated should be disposed safely.

III. Integrity of Water within the Distribution System

Even after treatment, there are several factors within the WDS that lead to deterioration of water quality. Ageing pipelines, pipe-breaks, or leaks make a WDS vulnerable to contamination. The water supplied through distribution networks provides a favourable environment for bacteriological growth due to corrosion, sediment accumulation, long residence times, the presence of nutrients, etc. Such detrimental effects undermine the quality of water post the water treatment plant. WDS integrity is, thus, of primary concern to ensure maintenance of satisfactory water quality during distribution. Water quality monitoring and contamination event detection systems throughout the WDS pose a technical challenge to every water utility but are essential for ensuring safe drinking water supply in addition to ensuring WDS integrity. Regular maintenance and cleaning protocols can help prevent unexpected deterioration in water quality. Transmission mains should also be subject to such protocols, to ensure that all pipelines preserve the quality of the treated water as much as possible.

Disinfection byproducts (DBPs) are formed by the interaction of Natural Organic Matter (NOM) present within the WDS with the residual chlorine in water. Common DBPs are trihalomethanes (THMs), haloacetic acids (HAAs), etc. Considering the potential carcinogenic effects of DBPs on humans, DBP control should be a priority for water boards. Alternative treatment processes like UV radiation and ozonation etc. show promise in tackling DBP formation. These treatment technologies are associated with significantly higher treatment costs. A thorough cost-benefit analysis, which would enable selection of such alternate treatment methods, to ultimately meet the goal of improved water quality is required.

Chlorine dosages in water treatment plants are ascertained to ensure the presence of an optimal residual chlorine concentration within the distribution system. Generally, a lower limit and upper limit (0.2 mg/L and 0.5 mg/L) is provided for these residual concentrations, such that the chlorine concentration is sufficient to account for bulk and wall reactions but not high enough for the formation of disinfection by-products (DBPs) which are carcinogenic in nature. However, in large distribution networks (with high water age) or old pipelines (with extensive bio-film deposits) the residual chlorine concentration falls below the desired limit, jeopardising the quality of the supplied water. Booster doses may be necessary at intermediate locations in the distribution system to maintain optimal residual chlorine concentrations.

IV. Water Safety

Any failure to ensure a safe drinking water supply is a significant public health risk, which leads to higher healthcare costs and lower economic productivity. To avoid such failures, the World Health Organisation's (WHO) Guidelines for Drinking Water Quality (GDWQ) lays out a detailed Water Safety Plan (WSP). This Plan provides comprehensive management strategies to prevent disease outbreak by protecting catchment-to-consumer water flow from contamination, by optimising treatment plant performance, preventing contamination during storage, distribution, and handling of the treated drinking water. Figure 4.19 provides a pictorial representation of the safe drinking water supply framework, which includes WSP.

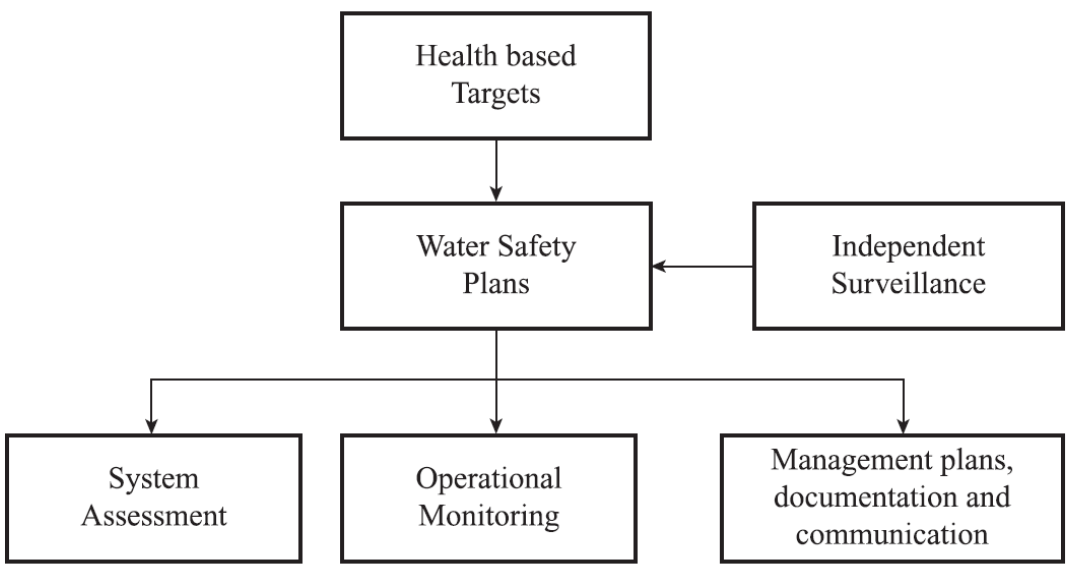


Figure 4.19: Holistic framework (including Water Safety Plan) for ensuring safe drinking water supply

After the 9/11 attacks, the global water community has become increasingly aware of the threats of bioterrorism and cyber-attacks through water systems. Increased automation in control and distribution opens up urban water systems to such external threats, several of which were previously unheard of. Such situations bring up challenging questions on the optimal placement of water quality sensors within a WDS, to minimise the risk and time of exposure to any accidental/intentional contamination. Cybersecurity protocols should also be updated continuously, to prevent unscrupulous elements from gaining control of urban WDSs.

V. Used water Treatment, Reuse and Recycling

Used water treatment is a very important component of IWRM. A good water supply system should be complemented with a robust used water collection and treatment system. Sustainable water use can be achieved only if cities in India resort to minimal (or near-zero) water wastage systems. Many present-day systems are a "disposal-based linear systems", where untreated sewage is disposed into surface or groundwater resources, rendering them polluted. As opposed to that, a closed-loop treatment system is recommended, which promotes used water reuse, recycling, and recharge. Benefits of safely recovering and reusing used water include a reduction in effluents to water bodies, and the opportunity to enriching soil with valuable organic matter. The nutrients in reclaimed water can replace equal amounts of fertilisers during the early to midseason crop-growing period. Level of necessary treatment of used water depends on its intended use: secondary treatment may be sufficient if the reclaimed water is to be used for agricultural or cooling purposes, while tertiary treatment is recommended for sanitary or gardening use of the recycled water. In many highly populated cities such as Tokyo (Japan) and Seoul (South Korea) there are in-plot treatment systems which reclaim the used water from houses and use it for toilet and urinal flushing purposes. This mode of water/used water usage is generally termed as dual water supply. Some countries depend on treated used water for irrigation purposes as well. There have been demonstrated economic benefits of using used water for irrigating non-edible crops like mulberry floriculture. Treated used water has also been used for recharge of groundwater aquifers with adequate safeguards. Usually used water treatment involves collecting the used water in a central, segregated location (the used water treatment plant) and subjecting the used water to various treatment processes. Decentralised systems are also a feasible alternative at certain locations, although the environmental impacts should be thoroughly assessed.

VI. Creating New Source of Water

Based on a city's requirement, tertiary-treated water can be provided with an additional treatment step to elevate its quality to drinking water standards. Allowing natural flow through long rivers or channels, percolating through soil to groundwater aquifers or treatment methods like RO or ultra-filtration (or a combination of some of these methods) provide this additional level of treatment. The resulting water is ready to be blended with freshwater and used for drinking purposes. This way, recycled water need not be assigned only for secondary uses but can also become a part of source of water. Awareness campaigns may need to be conducted by city authorities to help remove psychological barriers related to the use of recycled water for drinking.

VII. Urban River Rejuvenation

The water quality of most urban rivers is in deplorable condition, primarily due to indiscriminate dumping of untreated wastes, both solid and liquid especially industrial wastes. Rejuvenation of such rivers not only improves local environment and ecology, but also provides favourable locations for recreational activities. To prevent further contamination of urban rivers, drainage systems should be revamped, and thorough sewage treatment should be ensured. Solid waste collection mechanisms should also be improved to reduce indiscriminate dumping into rivers. In addition to centralised UWTPs, in situ treatment technologies can be employed if found feasible and cost-effective. Discharge of treated used water into urban rivers can assist in replenishment of the flows, while boosting the environment and ecology.

Some of the important considerations of an urban water supply system (typical components illustrated in Figure 4.20 are described below:

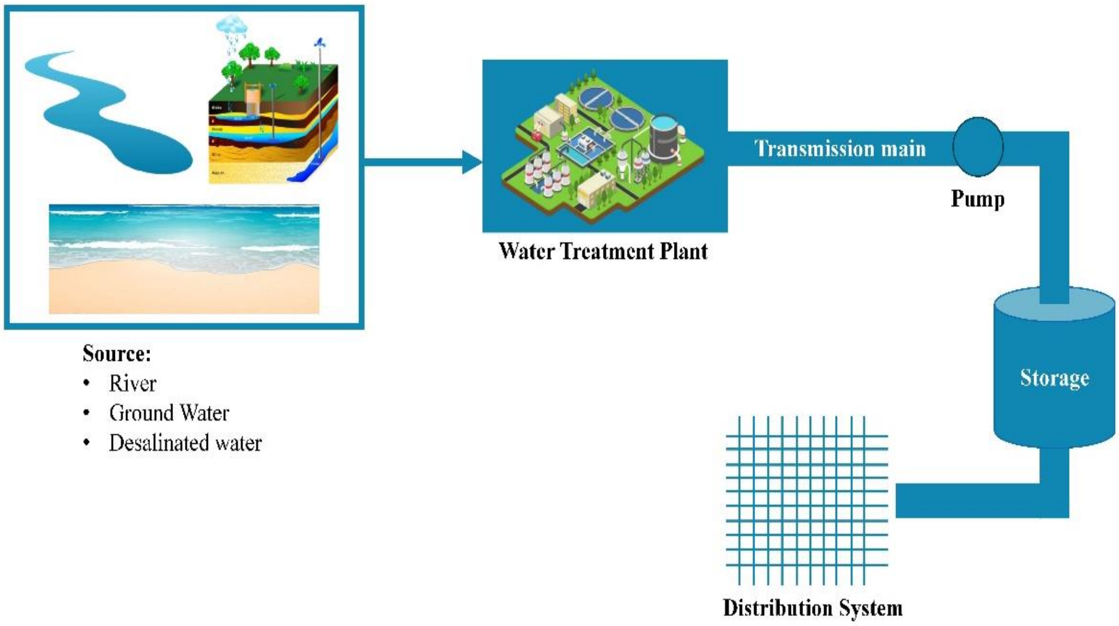


Figure 4.20: Various components of an urban water supply system (Pump if required)

4.13.6.3 Associated Infrastructure

I. Transmission and Storage

The transmission mains are the lifeline between the source, the water treatment plants, and the city distribution. Thus, transmission mains should be designed to be able to reliably transfer water long-term, with fail-safe features. Regular maintenance and monitoring of the transmission mains can ensure that the primary water supply source is always available. Storage tanks are an equally important part of the water supply system. Water storage provides a flexibility in intermittent system performance and adds a cushion to water availability in case of sudden shortages. The integrity of storage structures has a direct effect on the quantity, as well as quality of the water present. The elevation of the storage tanks also has an impact on pumping costs, so decisions on tank placement should be made on techno-economic principles.

II. Water Distribution System

WDS operation is inherently uncertain due to random change in demand, flow pattern, pressure head, and ageing. Hence, knowledge of reliability, resilience, and vulnerability is important in understanding the system behaviour for different scenarios in a better way. These three aspects can be defined as the probability that the system can provide the required flow rate at the required pressure (or how likely a system will fail), how quickly it can recover from failure, and the severity of failure respectively. For any WDS, reliability, resilience, and vulnerability calculations by developing/using a proper method are of utmost importance to operate the system in an efficient manner in any situation. Random analyses, employing ample historic data, should be used to predict the probable real time performance, as well as future trends under various scenarios of population growth, urban expansion, climate, and lifestyle changes, etc.

The reduction of NRW should be a concerted and continuous effort. Approaches based on district metering areas, the utilisation of network-based sensors for flow and pressure management and implementing cluster or cohort analyses (utilising network leak data) are now established measures to identify priority areas network improvement and to determine the priority region for network rehabilitation. Such an analyses will help in utilising the funds for the priority network and to gain maximum return in terms of NRW.

4.13.6.4 Efficiency in water use at every stage

In addition to the abovementioned practice, effective water distribution and use can be achieved by ensuring water efficiency at every stage. This includes the various transmission mains and water supply pipes. Additionally, the use of water saving fixtures at points of use can also significantly reduce per capita water use.

Canal and agriculture may be removed as beyond the scope; Leakage/UFW to be mentioned among the demands.

4.13.6.5 Data Requirements

a. Physiography

Physiography is the study of physical features of earth's surface. It includes information related to region's elevation details, soil type, and vegetation details. The following aspects are to be studied:

- Natural features
- Elevation profile
- Land use and land cover

It is to be understood that most of our Indian Cities and surrounding areas are undergoing rapid urbanisation and hence considerable land use changes, resulting in drastic changes in surface runoff as well as recharge characteristics can take place. These physiographical changes expected to occur in the near and remote future should also be accounted for while developing IWRM plans.

b. Hydrometeorological Details

Prior to preparation of IWRM, it is important that the hydrometeorological details are collected and analysed. The hydrometeorological details are combination of hydrological details such as details of precipitation in storm events, land and atmospheric water interaction and meteorological parameters such as rainfall, temperature, and relative humidity. Across the cities, there are very wide variation in rainfall patterns. These variability patterns can be effectively captured by increasing the spatial density of monitoring stations. In other words, higher the number of monitoring stations with higher frequency of data gathering, better is the analysis, especially for storm runoff calculation.

c. Geology Information

It is recommended to study the geological details of study area in conjunction with groundwater assessment. At the minimum, following aspect should be assessed:

Geology of area

- Geological age
- Stratigraphic units
- Lithological characters
- Water bearing characteristics

4.13.7 Stage II – Developing Dynamic Operating Model

4.13.7.1 Dynamic Operating Model (DOM) System and Telemetry

The Dynamic Operating Model (DOM) will analyse different types of data streams, to understand and optimise the water system. The raw data comes in many forms, including that from deployed remote sensors (flow, water quality, etc.), external organisations (meteorological data, etc.), and existing databases created internally or by other organisations (agricultural data, historic flow data, etc.).

Focus of the DOM system will be on optimisation of system for quantity, quality and cost, and although aspirational in nature, it is recommended to include DOM along with Instrumentation Plan and Telemetry in the IWRM Plan.

The DOM system monitoring framework will drive the identification of data required, and the consequent selection of sensor types and locations. Once the sensor types and locations are identified, the next phase will be the development of an instrumentation plan and a telemetry plan.

Development of an Integrated DOM Dashboard

It is important to establish the baseline for current water availability, prior to implementation of any water supply scheme. Flow and level monitoring are critical, as this information will provide data on additional water that has been made available through these schemes.

The primary goal of the integrated DOM dashboard is to collect data from the different source data streams and convert it, using the decision support system, into actionable information. The conversion of data streams to actionable information involves algorithms to process the data, and presentation of the information in a manner that is intuitive to an operator. The information is therefore presented on an interactive dashboard that provides clear visualisation of the information with drill down access to the underlying detailed data.

Development of the visualisation dashboard will involve discussion with stakeholders, as the different components of the system are designed and constructed. The information that is developed and incorporated into the DOM dashboard must help the decision makers to understand the critical information for the system, and the best mechanisms for presentation of the information in an intuitive manner. The dashboard will include GIS-based, graphical, and numeric presentation methods as appropriate.

The data streams will be collected in different databases for analysis as needed, either historical analysis, or as part of the dashboard presentation (i.e., graphical analysis of data points over time). Each of the data streams will have their own database management tools. The data sources will go through a multi-tiered data management architecture which will validate and provide QA/QC for the data.

Data will be sent via open standards such as web services or leveraging direct database connections as appropriate to support expansion of the system into the future.

Communication and visualisation of information to staff at all levels is very important to ensure that all staff have the most up-to-date information available to them. Secure internet communication methods can be used to rapidly push information to all partners.

Once the data has been processed into information, the information will be converted into a number of Key Performance Indicators (KPIs) that the management staff can use to evaluate the overall performance of the system against set goals and objectives.

The three areas of actionable information that will be derived from the dashboard for the operational optimisation tool are water quantity, water quality, and operational efficiency. This has to be done at necessary space/time intervals to understand the system dynamics better. The DOM will become increasingly more valuable as the system starts to develop and real-time data becomes available to allow system optimisation to occur.

Bangalore Water Supply and Sewerage Board (BWSSB) has a functional and fully updated GIS portal for water supply, asset management, real-time monitoring of used water treatment plant operations. This portal serves as a remote tool for understanding the ground reality and making informed decisions. In association with the Indian Institute of Science Bangalore's researchers, this portal is being rebuilt to enable real-time acquisition of big data, cleaning, analytics, and archiving. This portal will eventually serve as a one-point interface for complicated decision-making through improved assessment and visualisation of the ground truth of water systems. The improved portal will be opened for public viewing in the near future. Similar platforms have to be designed for used water treatment and reuse, storm water, etc. and integrated with drinking water portals to get a holistic picture of water resources management for cities or towns.

4.13.8 Stage III – Development of IWRM Plan

The final stage, preparation of an IWRM Plan, is driven by preparation of a detailed water balance. Preparing a detailed water balance is imperative for any city. Water balance calculations refer to a detailed break-up of the various sources of available water (from surface water or groundwater sources, rainwater harvesting, recycled water etc.), as well as the various demands (residential, institutional, industrial, horticulture, firefighting, ecology/environment, etc., and the water supply that remains unaccounted for). Under changing climate, growing populations and rapid urbanisation, the changes in each component of the water balance analysis should be updated for more realistic decision-making. This will aid in the development of various scenarios a city's water system may face and increase preparedness towards unforeseen situations Figure 4.21 shows the various components to be accounted for in a city's water balance, namely, water resources, water storage units, water treatment units, water demand, used water generation and treatment capacities. Details of a city water balance plan can be found in the subsequent chapter.

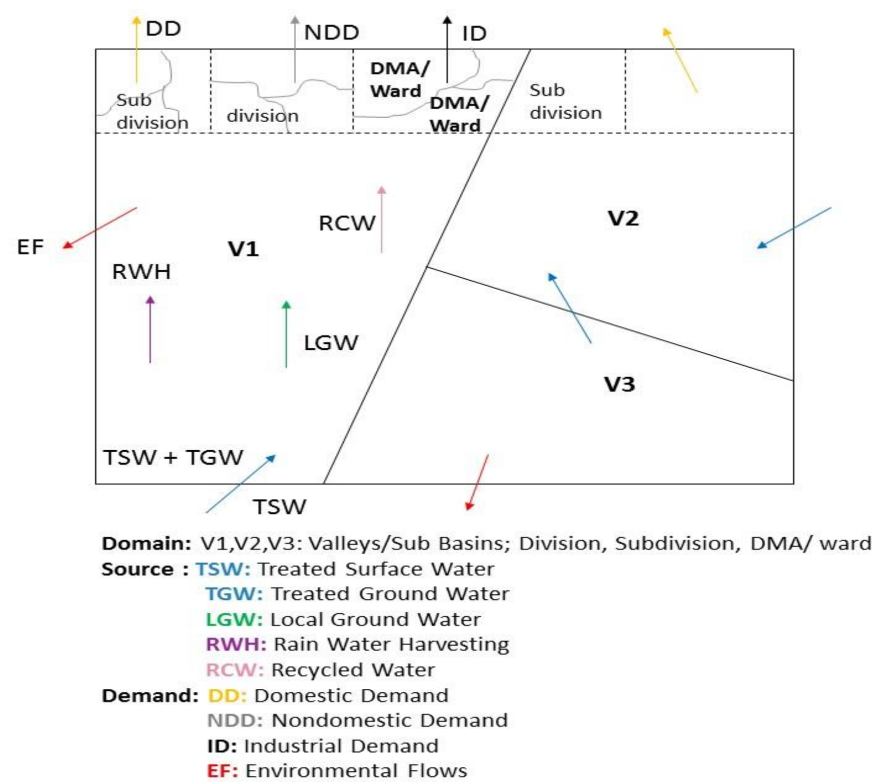


Figure 4.21: Movement of different components of water within a city

4.13.9 Water Resources Assessment- Availability and Demand

The water demand estimates and planning for efficient use of water is important when there is a limit to its availability. A good understanding of future water use will help to optimise plan for future water supply, infrastructure construction and system operations. The following are the types of water demands that needs to be estimated while preparing an IWRM Plan.

- Domestic
- Non-domestic
- Commercial
- Recreational
- Industrial water demand
- Institutional water demand
- Horticulture water demand
- Firefighting water demand
- Urban irrigation water demand

It is important to note that there are guidance provided by CPHEEO Water Supply Manual (Central Public Health and Environmental Engineering Organisation (CPHEEO), 1999), however, these values should be taken as guidelines, and the project specific water demands must be developed after judicious assessment of water requirements based on water availability, opportunity to utilise the recycled water, and applicable water demand management measures. The City Development Plan (CDP) needs to be kept in focus while developing water demand plan to avoid potential conflicts.

The typical planning horizon for water demand projection should comprise of short term (up to five years), midterm (up to 15 years), and long term (up to 30 years). It is important to note that this planning horizon is for guidance purpose, as different cities have different population dynamics that are dependent upon economic, environmental, and societal factors, however, the planning horizon should be such that it provides direction for the city to plan ahead and identify a diverse and resilient water portfolio, with enough leeway to make course correction as city continues to develop.

4.13.10 Potential for Demand Management

Demand management is a critical part of any IWRM Plan. As opposed to the supply side solutions, the cost for implementing demand management measures is modest to relatively low. In the long term, effective demand management would enable best practice management of overall water supply and infrastructure. There is sufficient scope to suggest ways to reduce the actual water consumption rates by using world's best management water practices such as:

- water-saving fixtures/devices;
- behaviour changes and social awareness/education programmes;
- leakage detection and repair;
- minimisation of NRW losses;
- increasing the reliability of supply/reducing local storage;
- economic pricing with escalating blocks water tariff.

The 150 LPCD is a guideline or aspirational design figure. In actual practice, water usage should be significantly less and therefore reducing the capacity of water resources to find, minimise the size of water infrastructure and reduce the capital cost and operating costs. It needs to be kept in mind that total of 150 LPCD can be met partly through local supply such as harvested rainwater, use of recycled water, etc. Water tariff is a sound tool to reduce water consumption.

4.13.11 Measures to Minimise Water Consumption

Any city could realistically achieve a domestic water demand of about 135 LPCD across whole city, but only after implementing the following measures:

- Higher plumbing and piping standards to prevent leaks
- Strict construction standards, contract supervision and leak testing
- Leakage and NRW monitoring and correction
- Water pricing to deter the waste of water and leakages, with incentives for lower consumption
- Implementation of control devices at critical points to bring in equity
- Strict controls and enforcement of water use rules
- High levels of public awareness campaigns and education

Given the low level of effort desired to achieve good results and cost savings with water demand management, it is essential that a city adopts such best management practices.

4.13.11.1 Estimate of Potential Water Savings

Residential demand management is achieved by reducing the per capita water consumption. As the city plans for its future water supply, it is prudent to use a guidance value of 135 litres per day, which is sourced by different water sources in a portfolio (such as surface water, harnessed rainwater on catchment scale, recycled water). As the city approaches towards 24×7 pressurised water supply system, the city can then implement strong water demand measure to promote lower per capita consumption. For instance, limiting the water to 120 litres per day has the potential to about 11% residential water over the Indian design standard. If provision for non-potable water supply is made say through recycled water, the drinking water supply should necessarily be brought down to 100 LPCD.

4.13.12 Infrastructure Requirements

Water demand management can be enabled through installation of water saving fixtures at points of use. No additional storage, conveyance, pumping, and treatment are required since the water saving fixtures control demand simply by limiting the flow of water from the tap or fixture without compromising on user satisfaction/efficiency.

There is no water infrastructure requirement for the implementation of water demand management as the water savings are made at the customer or user side. Nevertheless, the development and implementation of the following measures are needed as a minimum:

- Sound plumbing regulations
- Monitoring and enforcement of plumbing regulations
- Educational programmes and public awareness building
- Monitoring and management of NRW and leakages
- Reduction in storage volume at consumer end to reduce unnecessary storage

4.13.12.1 Operation and Maintenance Requirements

Operational and maintenance issues are mainly confined to the monitoring and enforcement of plumbing regulations and effective management and control of NRW and leakages both supply side as well as demand side. Information Education and Communication (IEC) activities are continuously required to reinforce the importance of water savings, good plumbing, and water use habits.

4.13.13 Institutional and Legal Considerations

Clear governance of the management of water infrastructure and services is required to realise the full potential and benefits of water demand management. As such, the responsibilities for plumbing regulation and enforcement needs to be clear and well designed and implemented from the onset. Responsibility of ongoing and effective public awareness building, and education is also an important aspect for successful water demand management.

The development of sound and best practice plumbing regulations and enforcement of such regulations is the key challenge with water demand management. Good governance is needed in order to realise the full benefits and potential of water demand management. For the public water infrastructure, community-based monitoring against leakages and pilferage can be introduced as governance model. The community should have large representation of women as well as girl students.

4.13.14 Urban Flood Management

Urban catchments are hydrologically quite complex due to the close interaction between natural and anthropogenic processes. It has been observed that the causes of urban floods are quite different, namely it is a consequence of insufficient drainage in response to a sudden high magnitude rainfall event, coupled with imperviousness and lack of flow space. According to the National Disaster Management Guidelines (National Disaster Management Authority, 2010), urban flooding is significantly different from rural flooding as urbanisation leads to developed catchments, which increases the flood peaks from 1.8 to 8 times and flood volumes by up to 6 times, when compared with undeveloped land space of same area. Consequently, due to faster flow saturation times (just a few minutes) flooding occurs very quickly (National Disaster Management Authority, 2010). Urban flood water, though conventionally let off into water bodies, can serve as a resource when harvested properly. With requisite quality control, urban flood water can be used for recharge of ponds and tanks, as well as for replenishment of groundwater aquifers.

The first step towards flood water harvesting is to improve the general understanding of the urban catchment i.e., exploring the natural and anthropogenic features that may contribute/alter the hydrology of the region. Extensive catchment analysis and mapping must be done using a variety of techniques, with the end goal of improving familiarity with the catchment features (Sahoo and Sreeja, 2017; Zope et al., 2015). This will also help generate a suite of catchment responses under various precipitation events, and also for projected climate change scenarios.

In a similar study done for studying and developing urban flood management measures for the entire Bengaluru City, Figure 4.22 presents Bengaluru's drainage map.

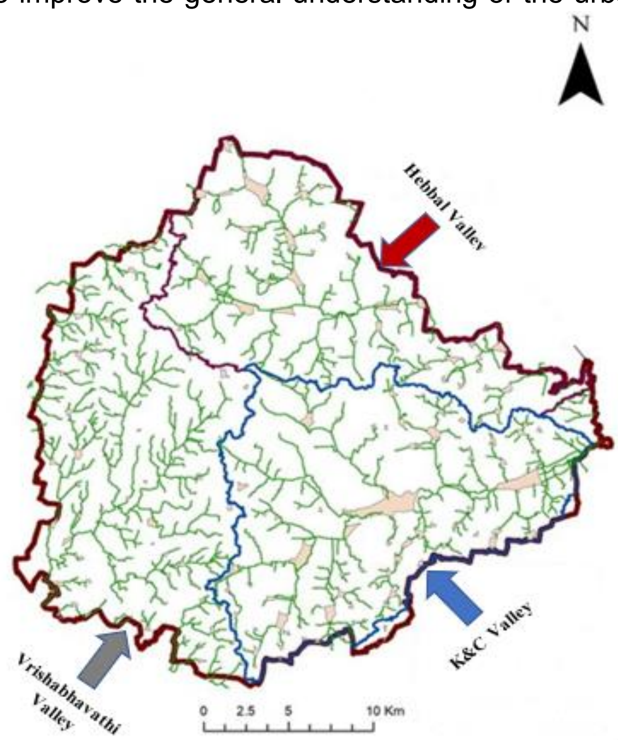


Figure 4.22: Bengaluru's drainage map

4.13.15 Guiding principles for developing IWRM plan

All water infrastructure projects are prepared based on specified goals, desired levels of services, and long-term aspirations. Establishing a set of guiding principles to represent these goals, level of service and aspirations help to describe the fundamental requirements that a project must overcome before it is considered worthwhile implementing or evaluating further. Guiding principles also help in framing the decisions to be made by defining the scope of the issue, the limiting conditions, and desired outcomes.

The guiding principles proposed for this project are summarised in Table 4.16. Each includes a description of the corresponding planning goal(s) as well as a note on whether it is a basic need or Aspirational goal.

- Planning Goals – This can be qualitative or quantitative. Measurements/Indicators will be used to evaluate if the principles are met. Solutions that meet this criterion will be considered for the project.
- Basic need or Aspirational – This helps to define the planning goals further. Basic goals are the key priorities to meet in order to successfully achieve the outcome of a sustainable IWRM plan. Aspirational goals are good to have in order to outperform the original goals of the project.

Table 4.16: Proposed Guiding Principles for the IWRM Plan

Guiding Principle	Planning Goals	Basic or Aspirational
Ensure social equity in terms of access to good water quality and quantity to sustain human activities.	All solutions must meet current regulatory requirements (for used water discharges, storm water management, potable water treatment, and fire demand).	Basic
Provide water infrastructure and services that are cost effective over a 30-year life cycle.	Long term financial cost (based on life cycle cost) for the integrated system is minimised.	Basic
Ensure maximum efficiency in using scarce water supply and financial resources.	Electrical power requirements for system components are minimised.	Basic
Develop in a sustainable manner and without compromising ecosystem.	Ecosystem (habitat and biodiversity) are protected or enhanced.	Aspirational
Balance capacity of potable and non-potable systems for steady and secure water supply.	Demand for both potable and non-potable water is met.	Basic
Minimise potable water use for non-potable purposes.	Non-potable water (captured rainwater, reclaimed water, or grey water) provides at least 50% of the traditional potable water demand.	Aspirational

Guiding Principle	Planning Goals	Basic or Aspirational
Maximise IWRM Plan flexibility to adapt to changing conditions over time.	Recommendations include options that are less susceptible to risks from changing conditions, and capable of retrofitting or upgrading to meet new conditions.	Basic
Facilitate public acceptance.	System components/strategies do not cause any public concerns about safety or reliability, Use of treated recycled water for blending.	Basic
Utilise state of the art principles and solutions, transparency at every level, and involvement of stake holders.	Strategies and recommendations are comparable to best practices in other places.	Aspirational

4.13.16 Financial sustainability and stakeholder engagement

To ensure financial sustainability for an IWRM Plan, the following three key measures should be considered:

- ensuring that revenues cover all operating expenditure;
- delivering capital programmes without incurring an unsustainable debt burden; and
- reducing the existing financial deficit.

To achieve these measures, a three-pronged approach will focus on optimising expenditure, boosting revenue, and smart financing (Figure 4.23). Effective governance is the necessary foundation that enables all these outcomes.

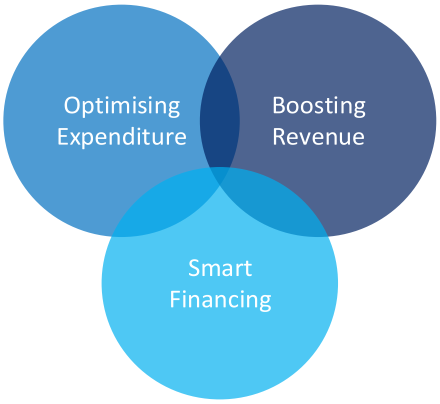


Figure 4.23: Strategy to achieve Financial Sustainability

4.13.17 Challenges in financing the water and used water sector

Water supply and sanitation services provide both social and economic benefits. Water is essential for basic health and sanitation but is also an important enabler for many sectors of the economy, agriculture and manufacturing for example. Because of this dual benefit, stakeholders have varying opinions as to whether water is a basic right to be provided based on social grounds, or whether it should be supplied and charged for based on commercial criteria. Often, an ineffective compromise results, with sources of funding, and expenditure and revenue plans unclear and inconsistent.

Also, in comparison to water, it is arguably harder to recover costs for used water services from consumers. While there is a strong incentive to connect and pay for a clean and adequate water supply to fulfil basic needs, the incentive to pay for proper collection and treatment of the resulting used water is less obvious to the individuals.

With the paradigm shift in viewing used water as a resource than as a waste however, there are stronger arguments and incentives for paying for used water services as a raw material for recycled water production. Financing for water supply and used water management services in general comes from two sources: tariffs, taxes, and transfers (the 3Ts), and market-based finance. Figure 4.24 explains these forms of finance.

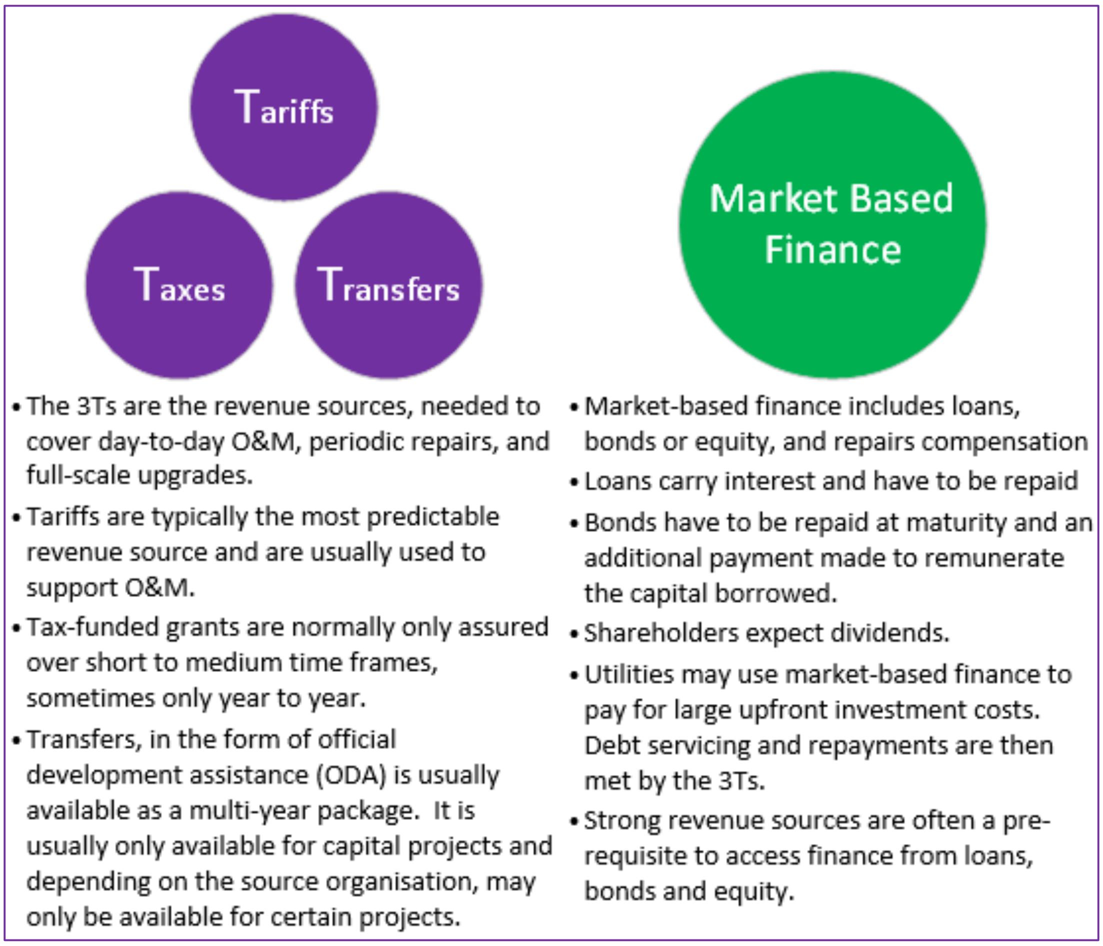


Figure 4.24: Sources of Finance

4.13.18 Creating Financial Sustainability

Considering the aims for financial sustainability, the following principles can be implemented for an organisation when planning future projects:

- In the short- to mid-term, user charges can be used to finance operation and maintenance related expenses (including debt service), routine capital expenses (replacement of existing assets at end of life), and NRW reduction projects.
- The financial planning shall be such that water utility/organisation must be able to recover the capital cost (including depreciation) in the long term, for organisation's financial sustainability.
- Projects that provide social and environmental benefits without tangible financial returns to water utility/organisation shall be funded using non-debt instruments (i.e., grants and contributions) wherever possible.

The following measures can be adopted to create financial sustainability.

4.13.18.1 Optimising expenditure

For most of the water utilities, the funding required to deliver existing operations typically exceeds the revenue it generates through its water tariffs. To close this gap, water utility must identify and address the ways it currently delivers its O&M activities. It shall also seek to develop and deliver its capital schemes more efficiently, thereby reducing the amount of funding it must generate through debt.

A water utility's major sources of operating expenditure are listed in Table 4.17, and it is recommended that the water planners look critically at these expenditures to identify opportunities for OPEX reduction.

Table 4.17: Major operational expenses in a water utility

Category	Items
Power	• Pumping of water (surface water and/or other water sources). • Power use at WTPs. • Power use at used water pumping stations. • Power use at UWTP.
Administration and management	• Labour costs at headquarters. • Pension commitments.
Repair and maintenance	• Labour costs associated with O&M. • Materials and equipment for O&M activities. • Pollution cess charges. • Royalty charges
Debt service	• Interest payments. • Guarantee commission charges (some funding agencies take certain percentage as guarantee charges on outstanding loans).
Depreciation	• Depreciation in the value of fixed assets.

4.13.18.2 Maximising Revenue

A water utility's revenue from tariffs covers approximately ranges from 40% to 60% of its operating costs. A water utility shall make the case for altering tariffs and increasing the total tariff revenue by:

- Consulting with customers to understand future demands, the outcomes customers expect, and the amount they are willing to pay. This is likely to vary significantly between different customer types (not necessarily aligned to water utility's current customer distinctions).
- Establishing effective accounting systems so that the real costs of services can be accurately established and then tracked.
- Clearly outlining a long-term strategy for improving services, followed by regular updates and opportunities for customers to engage with the water utility.
- Seeking to establish a clearer link between customer costs (i.e., the tariff) and the benefits that customers receive. This could be achieved by, for example, defining outcome measures and service commitments that water utility shall use its revenues to deliver, and then regularly reporting on performance. It is critical that any tariff increase is accompanied by better service.

- Considering promoting a mechanism of indexation which will enable revenues to grow in line with costs.
- Providing a mechanism for customers to hold water utility to account for its service commitments.
- Planning for potential future regulatory landscapes, such as an independent regulator that shall assess water utility's performance and use this to recommend changes to tariffs. This form of regulation exists in the UK.

4.13.18.3 Financing Options

Effective governance is a pre-condition for smart financing. In particular, governance structures must create clear core functions for policy formulation, regulation, asset holding and service provision. Capital projects in water and used water require large investments over short periods of time, creating assets with long lives which in turn require regular investment in operations and maintenance. A variety of alternative finance options for capital projects exist, each of which has specific advantages and disadvantages. In practice, capital investment and recurrent costs tend to be financed in different ways. Investment is typically funded by grants, loans, and bonds whereas recurrent spending is often reimbursed from tariff revenues and subsidies. Ultimately, all expenditure has to come from the 3Ts – tariffs and other user contributions, tax-base subsidies or transfers.

Many global cities have accumulated substantial debts through loans and bonds and in some extreme cases, Detroit for example, these cities have filed for bankruptcy. Debt financing shall only be considered as one part of a sound financial strategy.

In order to finance its projects in a manner that supports its aims for financial sustainability, the water utility must:

- clearly identify the projects it needs to deliver to achieve its target service levels. Setting the levels of service is therefore also a key task;
- accurately estimate capital and operating cost requirements. This must consider the full life cycle of asset costs, including operation and maintenance, decommissioning and disposal;
- evaluate alternative capital funding mechanisms; and
- assess impact of covering additional operating costs through tariffs.

Figure 4.25 presents this methodology.

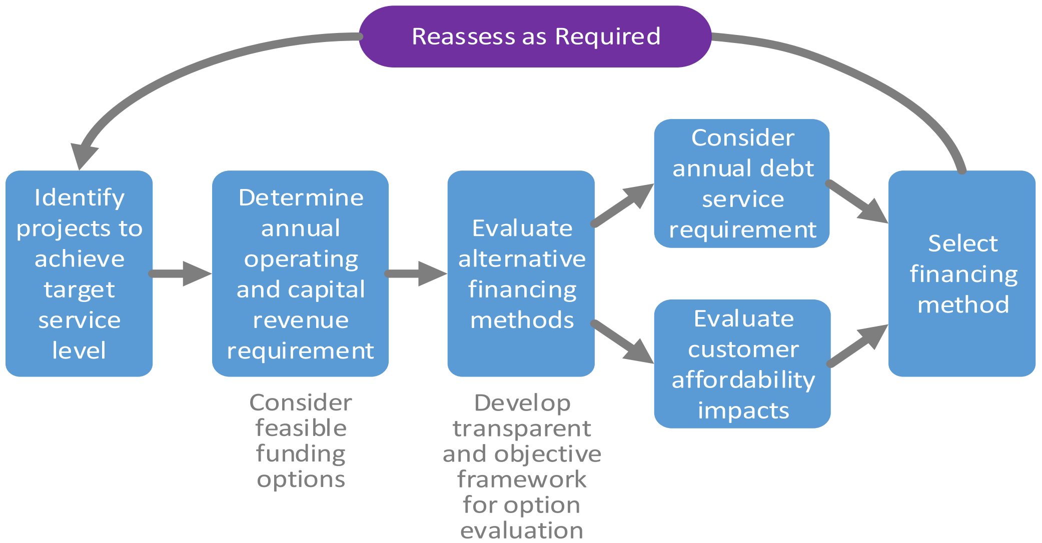


Figure 4.25: Methodology for planning project financing

4.13.19 Stakeholder Identification

Stakeholder identification is a critical component of an IWRM Plan. The stakeholders can be distributed in to two categories:

- Primary Stakeholders – who have a direct interest or influence on the IWRM Plan; it includes:
  - Water Board/Utility, General Public, Municipal Organisations, Land/Building Development Authorities, Regional Development Authorities, Water Resources Conservation /Management Authorities, Pollution Control Boards, State Government, Government of India
- Secondary Stakeholders – who are not responsible for specific activities that relate to water management, but they do have an indirect interest in IWRM Plan; it includes:
  - State Water Supply and Drainage Board, State Urban Infrastructure Development and Finance Corporation, State Industrial and Infrastructure Development Corporation, State Agriculture/Horticulture/Aquaculture/Energy Depts, Finance Bodies, Research Institutes, Non-Governmental Organisations, Suppliers and contractors, other utilities

4.13.19.1 Strategy for Stakeholder Engagement

Effective stakeholder engagement requires a comprehensive and inclusive strategy that seeks out engagement and input from a broad range of stakeholders (government bodies, industries and businesses, funding organisations, regulators, community groups and the public). The strategy must also adapt and evolve at different stages in the lifecycle of the IWRM Plan, and a given project, from planning, through design and funding to implementation. At each of these stages, the extent to which a specific stakeholder should be engaged may change, as may their specific interests.

If a comprehensive strategy is not followed, stakeholder engagement becomes a risk leading to a largely reactive process implemented in an authoritarian manner, often in response to crises. In contrast, forward-thinking organisations now consider stakeholder engagement as an early, interactive, and inclusive approach which allows them to identify common ground, optimise options, and deliver mutually beneficial outcomes.

Organisation’s stakeholder engagement strategy for a given initiative should comprise a series of stages, as presented in Figure 4.26**.**

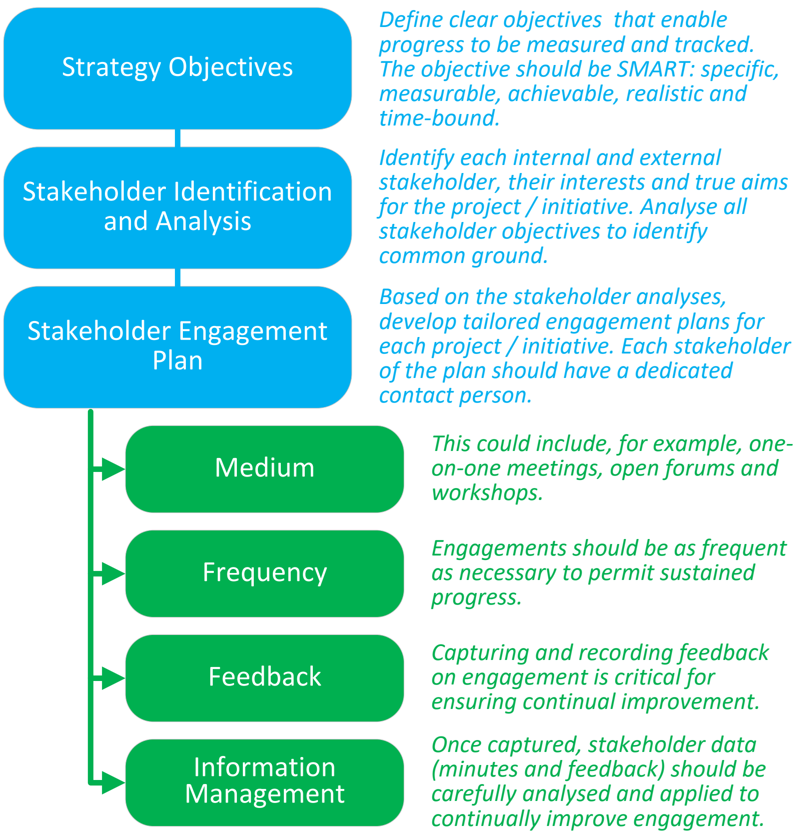


Figure 4.26: Stakeholder Engagement Approach

4.13.19.2 Approach and Format for Stakeholder Engagement

Successful and efficient stakeholder engagement requires a comprehensive and inclusive approach that involves institutional stakeholders as well as local communities.

For each project or initiative, there may be a slight variation in terms of targeted stakeholders, i.e., among the targeted audience for recycled water utilisation, more emphasis will be given to supply of recycled water for non-drinking purposes, industries and commercial establishments and they will be the targeted section, who can afford to pay for the recycled water services.

However, in most of the cases, the broader group of stakeholders will stay similar although the types of engagement, their locations times and frequency, and mechanisms for capturing feedback might change.

Some of the key items constituting the approach towards engaging the stakeholders are as below:

- It is important that each key stakeholder type should be assigned a specific contact person.
- To maximise beneficial outcomes, effective planning of engagement guidelines, as well as early stakeholder contact are key requirements.
- Guidelines for engagement should promote principles of honesty, trust and integrity and include transparency, respect, and partnership, ensuring that stakeholders are not judged for their values and that common ground is established.
- In doing so, organisation should be transparent about its own values, interests, and expectations at all times.
- It should be acknowledged from the outset of an engagement programme that there will often be disagreements between stakeholders. These disagreements are healthy debates, and their resolution helps to balance and optimise project outcomes.

- Disagreements should not be allowed to negatively affect stakeholder relationships and efforts should be made to ensure that all stakeholders remain receptive to each other's ideas.

Table 4.18: Means of Institutional Stakeholder Engagement

Format	Details
One-on-one meetings	Regular one-on-one meetings can be highly effective for bilateral discussions but are less suited to projects or initiatives that involve multiple stakeholders (such an approach does not foster transparency or collective progress). For regular one-on-one meetings, stakeholders should be assigned dedicated contact persons within Water boards in order to foster an effective working relationship.
Forums	Forums for selected attendees or open membership (public meetings) are effective for communicating information or educating about new concepts. The unstructured format may, however, be less suited to collective planning of projects. Forums could be held virtually to boost attendance however such events require careful management to ensure attendees remain engaged throughout.
Focus groups	Focus groups require a clear agenda, attendance list and expected outcomes. With this planning in place, they can be effective at collectively developing ideas and reaching consensus about important issues.
Working groups	Working groups are ongoing collaborative initiatives between multiple stakeholders that provide a platform for sharing information and co-ordinating research. Singapore's WaterHub is an excellent example of this concept. The facility provides a venue for collaborative working and an avenue for networking within the broader water industry, locally and internationally.
Questionnaires	Questionnaires can be a useful means of gaining an early understanding or appreciation of stakeholder interest and/or concerns. They should be followed by direct engagement to collectively develop ideas and solutions.

4.14 City Water Balance Plan (CWBPs)

Typical water scenario planning in urban areas emphasises water treatment, supply and subsequent collection and treatment of used water. This kind of an approach fails to account for the revenue potential of treated used water reuse, or even NRW. Clearly, a comprehensive accounting of all possible pathways for generation, use and reuse of water is required, which will enable simultaneous reduction in water wastage, as well as maximum revenue recovery. Additionally, such water balance calculations aid in the development of robust water policy, water management approaches and prudent investment decisions (Bahri, 2012). As cities grow in size and complexity, water balance modelling promotes maximised water reuse and minimises dependence on imported water (Barton et al., 2009).

City water balance plans (CWBPs) are generally prepared for a base year (e.g., 2020), with relevant projections for an intermediate year (usually 15 years subsequent to base year, i.e., 2035) and a design year (30 years subsequent to base year, i.e., 2050). The CWBPs may have various formats which are enumerated and described below:

i. City profile and demographic data: This consists of a self-explanatory CWBP format, which also contains GIS city map showing city boundaries, zones, wards, etc.
ii. Water sources: As outlined earlier, redundancies in a city's water sources are crucial for ensuring reliable and uninterrupted water supply to its residents. Data from each of these sources has to be carefully collected. If a source is located outside city limits, the water taken should be considered as borrowed water and should be accounted as such in the water balance.
iii. Urban water bodies: All existing urban water bodies (including surface and groundwater sources) should be mapped and their corresponding contribution to the city's water consumption should be ascertained.
iv. Water supply: The present demand, and gaps in future demand at the intermediate and design year should be assessed. These gaps should be mapped in city maps and accounted for in future planning or expansion projects.
v. Used water: Used water generation, collection, treatment and any gaps herein should be well reflected in the information collected. Thereafter, the availability of the treated used water for non-potable use in industries, institutions, commercial and domestic settings should be accounted for. The adequacy of used water infrastructure or any gaps therein should be reflected.
vi. City water balance abstract: A city water balance abstract can be prepared by generating data from all of the formats mentioned above. Losses, wastage, and other components can be estimated separately and entered.

It is of utmost importance that a baseline calculation for the supply and demand in a city/town, etc. is established. Though such baselines are established based on a thumb-rule approach, both static and dynamic approach can be considered for developing a water balance plan. Static approach is better suited for a broader level water resources planning wherein the water balance plan is to be demonstrated at an administrative level to plan for mid to long term water and allied infrastructure needs. Reference to this approach is presented earlier section. Dynamic water balance is suited from operational perspective when the data is available at DMA (or smaller geographical level) and needs to be integrated for managing water resources in real time. This approach can be utilised for developing spatial, temporal, and source wise water equity among different users.

If a city is seen as one of the demand points in a sub-basin, a sub-basin approach of hydrological calculations of source generation and various demands such as irrigation, drinking water, industrial water, etc. is the natural approach. However, these types of calculations could be only at a very large scale and could miss out on many aspects like rainwater harvesting, regeneration of used water, etc. Even if one were to do the calculations at sub-basin scale, care should be taken to downscale the same to the city scale and understand the interactions between, surface, ground water, rainwater harvesting and recycled water along with demand nodes such as service stations, command areas, etc.

To perform such calculations, very definitive water assets including storages, pipelines, pumps, etc. need to be mapped and made available in a GIS format. Along with this, the seasonal groundwater table data available need to be mapped as well. The volume of much fresh water that is supplied, recycled water that is generated and how much of it is used in the city, the volume of rainwater harvesting that is being carried out at local scale, indicate direct/indirect availability of water for both potable and non-potable purposes. This, along with the demands at various points in the city, indicates a balance of the water movement. It is also important to be aware of volume of water stored in the city, division/subdivision level and at ward level indicating the security in terms of storage. To show the movement of the above different components of water in a city, Figure 4.27 may be referred.

This figure shows the supply in terms of treated water, extraction of local ground water, rainwater harvested locally, and recycled water used locally. This has to be balanced with the demand from both domestic as well as nondomestic sectors. The details provided in the earlier sections about demand management can be referred, as necessary. Needless to say, such balance calculations need to be done at city/part of the city like division/subdivision/DMA/ward scale to get at the water balance scenario. These types of calculations have to be carried out periodically especially for monsoon, non-monsoon season. In all these calculations, environment/ecology needs to be kept in mind. Any city which imports less water from outside will ultimately be moving towards sustainable water resource management. Water balance calculations can be performed from the demand or supply side. Conventionally, a water supply board (supplier) is interested in knowing the various demand factions for each of its sources of water supply. This approach helps in better water apportionment for the supplier and detecting huge losses or unprecedented demands. Conversely, growing cities and towns may need to use values/projections of demands to predict required increase in source supply. This kind of calculations can be performed using demand-side water balance calculations. Due to the ever-increasing water demands and increasing complexity in apportionment problems of towns and cities, up-to-date and accurate water balance calculations are a necessity.

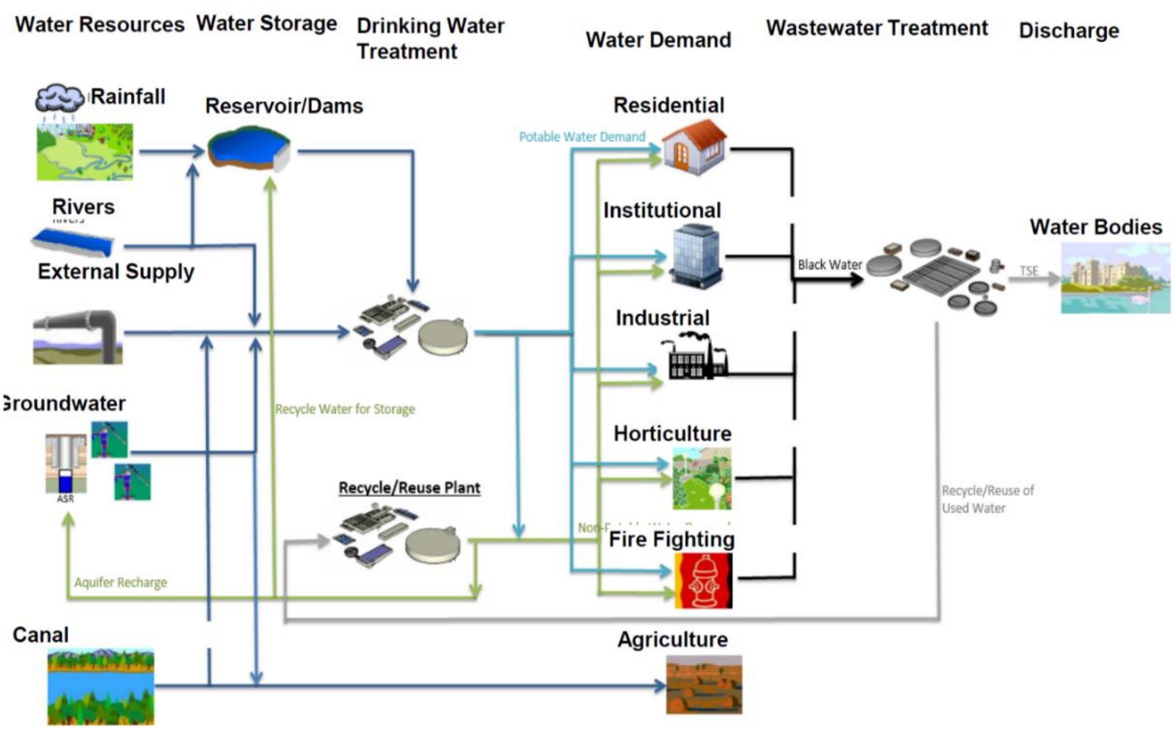


Figure 4.27: Various components to be accounted for in a city water balance

4.15 Use of GIS Mapping in Pollution Assessment of Water Resources

Drinking water source sustainability is coupled with water quality apart from availability of sufficient quantity of water. Key challenge is the contamination of water bodies, in the drinking water sources. Apart from contamination of water bodies, ground water based sources get polluted through leaching. Water pollution results from contaminant mix which can come from one of five main sources: sewage discharges, industrial activities, agricultural activities, urban runoff including storm water & hazardous hospital waste.

Sources of water pollution are either point sources or non-point sources. Control of water pollution requires appropriate infrastructure and management plans as well as legislation. Water resource mapping enables the identification of vulnerable areas that may face water scarcity or have inadequate access to clean drinking water.

GIS (Geographic Information System) is an essential tool for managing water supplies, as it helps to visualize and assess spatial data related to water resources & related attributes. While planning the water supply systems mapping on GIS platform need to be carried for identification of

- Location of water resources
- water quality of water resources
- location of untreated sewage / sewerage disposal points into the water bodies
- location of town waste water disposal points into the water bodies
- location of water based industries with attributes like type of industry, source of water, water consumption, quantity of waste water discharge, treatment system if any (capacity), treatment system details – primary treatment, secondary treatment etc., discharge water quality parameters, location of discharge into the water bodies.
- location of sewage treatment plant (if any) with following attributes 1) capacity, 2) year of commissioning, 3) maintenance agency, 4) major components of sewage treatment plant, 5) discharge water quality parameters, 6) location of discharge into the water bodies.
- Location of hospitals with following attributes 1) quantity of hospital waste generated, 2) type of disposal system, 3) Location of waste disposal, 4) discharge waste parameters 5) location of discharge into the water bodies.

Note: 1. While planning the project provision of Incineration can be considered. Possibility of PPP mode can be explored.

- IoT based monitoring systems to be used for constant monitoring of water quality parameters. The GIS system is also capable of generating alarms when any abnormal changes are found. The information so acquired can be vital for decision-making.
- Nature based treatment solution can also be considered based on techno-economic feasibility.

GIS platform will be helpful in understanding the spatial distribution of water quality parameters, identifying pollution sources, and implementing effective management strategies. The information is crucial for planning water supply systems.

4.16 Climate Resilient Drinking Water Solutions Through Sustainable Groundwater Replenishment Using Catchment Area Principle

Groundwater is still the primary source of agriculture and drinking water supply in urban & rural areas. Over the past two decades, the depletion of groundwater resources has been substantial, primarily due to uneven rainfall distribution, climate variations, hydro-geological conditions and excessive exploitation driven by pressing demands for self-reliance especially in agriculture and industrial sector, and also increasing population leading to the stress on ground water resources.

Due to over drawl and drought conditions, groundwater has been not compensated adequately with recharge of subterranean aquifers that store the water because of geological and physiographical conditions of the watershed. It has been observed that abnormal depletion of groundwater takes place during the extreme summer months affecting the groundwater based water supply systems. Apart from this groundwater quality is being deteriorating.

Hence the imbalance created of groundwater system now causing threat to groundwater sustainability. Hence there is a need to conserve the water to overcome the problem.

We realize the vital importance of water only when we do not have solution / option left with us to mitigate the problem is better & effective management and utilization of available water resources which in other words called as catchment area principle i.e. “Watershed / Spring shed Management”. The first Dublin principle which refers to management ‘across the whole of a catchment or groundwater aquifer' as being the most appropriate approach for a range of related land and water management issues. Since watershed represents a distinct hydrological unit, it is essential to use it as the basis for planning & implementation. With the concept of watershed management, a holistic approach is aimed at optimizing use of available water & other resources. A watershed is a geo-hydrological unit or an area that drains at a common point.

The basic purpose of artificial recharge of ground water is to restore supplies from aquifers depleted due to excessive ground water development

Recharge techniques normally address to following issues:

- To enhance the sustainable yield in areas where overdevelopment has depleted the aquifer.
- Conservation and storage of excess surface water for future requirements, since these requirements often changes within a season or a period.
- To improve the quality of existing ground water through recharge.

Basic requirement for Artificial Recharge projects are:

- Availability of non-committed surplus monsoon run off in space and time.
- Identification of suitable hydro geological environment and sites for creating sub surface reservoir through cost effective artificial recharge techniques using remote sensing / GIS as a tool.
- Need to develop various thematic maps based on the catchment area principle & with the spatial analysis using remote sensing & GIS tools, comprehensive drinking water security action plan can be evolved.
- The Jal Jeevan Mission guidelines suggest the use of Hydro-geomorphological maps (HGM Maps at scale of 1:10000) for identification of probable location for recharge of aquifers through site specific recharge structures.

Planning Framework

A. Project Area Identification using Remote Sensing & GIS Tools

Data Collection	Administrative Maps – District; Block; Topo-sheets; Geology Map; Hydro-geomorphological maps; Soil Map; Rainfall Details;
Geo-referencing of Maps	Geo-referencing & Integration of maps
Delineation	Delineation of watersheds boundary on 1:10000 scale using Topo-sheets
Thematic Maps	Development of Thematic maps – Geology showing faults & fissures. lineaments. soil etc.
Overlays	Generation of Overlay maps using thematic maps for analysis
Decision Making	Identification of suitable Project area for Recharge

B. Development of Comprehensive Drinking Water Security Action Plan using Remote Sensing & GIS Tools

Data Collection	Administrative Maps – Town / Village; Ward / Gram panchayat; Census Details; Institutional & Livestock Details; existing water supply status with present water supply levels; Water sources inventory with Water Quality Status etc
Geo-referencing of Maps	Geo-referencing & Integration of maps at 1:10000 scale
Delineation	Delineation of micro watershed boundary on 1:10000 scale using Topo-sheets
Thematic Maps	Development of Thematic maps –Town / Villages with Census details & water requirement; Ward / Gram panchayat; micro watershed; etc.
Overlays	Generation of Overlay maps using thematic maps for analysis
Water Budgeting	Water Demands assessment; rechargeable area; ground water balance; stage of ground water development;
Decision Making	Development of comprehensive drinking water security action plan showing site specific recharge structures

A CASE STUDY - GAJRA WATERSHED - BLOCK: PATAN DISTRICT: DURG

The Block Patan, District Durg - Chhattisgarh is water stressed block falls under the category of Grey block. The goal was to ensure drinking water as well as food security.

Methodology

The satellite data provides various different lithology, structural and morphological feature of ground which controls factors for ground water regime.

Groundwater potential mapping methodology involves acquisition of satellite data, use of visual interpretation technique, ground facts collections and final maps preparation of themes like physiography, slope, soils, drainage, geology, faults, fissures, lineaments, hydrogeology, hydro-geomorphology etc. In addition to ground water prospects, rainfall and climate, ground water fluctuation, G.W. quality, land use, data have been taken into consideration for final decision making of site selection of specific structure. Design guidelines for selection structure have been also considered while making final decision.

Study of individual thematic maps facilitate the analysis various parameters in respect of each theme under study.

The natural and logical associations of different parameters of one theme vis-a-vis those of the other are studied. With the logical explanation and guidelines some workable combination is worked out. Integration of thematic maps is done through GIS and results interpreted produce the action plan of proposed an appropriate specific structures site selection.

Results

With the implementation of identified site specific recharge structures in the said watershed, it has been observed that ground water table has been raised by approximately 25 meters with the construction of site specific recharge structures i.e. from 35-40 mbgl to 10-12 mbgl. No water quality problem reported in the area.

Recommendation

Accordingly, Department of Drinking Water Supply & Sanitation, MoJS, GoI has recommended that planning principles of Gajra Watershed should be replicated across the country to ensure long-term ground water security & sustainability.

4.17 Use of Hydro-Geomorphological Maps (HGM) for Identification of Ground Water Sources Location

Hydro-Geomorphological Maps (HGM maps) at the scale of 1:10000 which can be used to determine the probable location of the ground water availability with the use of GIS technology. This technology required to be adopted for identification of ground water based sources location in consideration with geo-morphology, hydro-geology, soil, faults, fissures, lineaments etc. should be supported by site specific recharge structures.

CHAPTER 5: PUMPING STATION AND MACHINERY

5.1 Introduction

Pumping of water serves a variety of functions in water supply systems, such as moving water from a source to a water treatment plant and from the treatment plant to the distribution system. High and low lift pumps are used consistent with the topography of land and location of the water treatment plant, whereas high service pumps are employed to discharge water under pressure to the water distribution system. Booster pumps are used to increase pressure in the water system. Recirculation and transfer pumps are used to transport water through a treatment plant. Vertical turbine (VT) pumps are employed in well pumping.

VT pumps are generally installed in source water intake pumping. Either VT or centrifugal pumps are commonly used for high and low service to lift and transmit water. VT pumps can also be used to move water to treated water transmission and distribution systems. Centrifugal pumps are popular because of their simplicity and compactness, low cost, and ability to operate under a wide variety of conditions. This chapter deals with the design of the pumping station, selection of pumps, their types, and characteristics, electric motors, their types, and characteristics, etc.

This Chapter also has important linkage and interdependability with Chapter 9 of Part B dealing with operation and maintenance of pumping station and pumping machinery because designs directly affect the effectiveness of operability and maintainability. Sample calculation for pumping machinery is enclosed in **Annexure 5.1**.

5.2 Requirements of pumping station

The subsections below detail the general requirements of the pumping station comprising intake/sump/other sources, pumps, and allied equipment in the pumping station.

Types of pumping station and source

The pumping stations are for housing pumping machinery powered by energy sources with required equipment and accessories housed in appropriate buildings to pump water at required points of interest such as water treatment plants or treated water to the consumer end. There are other locations also in a water supply system where pumping of water is required to increase pressure in a low-pressure zone or fill water in elevated reservoirs.

Types of sources and pumping stations are as under:

- River intake
- Intake in an impounded reservoir
- Intake in lake
- Piping intake from dam
- Sump and clear water pumping station
- Booster pumping station with sump and pump house
- In-line booster
- Borewell/tube well
- Dug well

Broad classification of pumping station

The size of pumping station depends on the quantity and quality of water and the head to which it has to be pumped. Since all the components are not required in every condition, the pumping station can be broadly classified as small, medium, and large as given in Table 5.1 below.

Table 5.1: Broad classification of pumping station

Sl. No	Size of pumping Station	Quantity of water Pumped in MLD
1	Small	Less than 25 MLD

Sl. No	Size of pumping Station	Quantity of water Pumped in MLD
2	Medium	25 to 125 MLD
3	Large	Above 125 MLD

Components of a Small, Medium, and Large Pumping Station

Sl. No.	Component	Small	Medium	Large
1.	Site and location of pumping station			
a.	Inlet channel	✓	✓	✓
b.	Screen or rose pieces/Drum screen/inlet strainer	✓	✓	✓
c.	Pre-settling tank/silting basin	✓	✓	✓
d.	Sump wells	✓	✓	✓
e.	Pump house	✓	✓	✓
f.	Pumping machinery	✓	✓	✓
g.	Suction and delivery piping system	✓	✓	✓
h.	Water hammer control device	✓	✓	✓
i.	Clear water reservoir			✓
2.	Electric substation and substation building			
a.	Metering panel	✓	✓	✓
b.	Transformers and transformer yard		✓	✓
c.	MCC panels, etc.	✓	✓	✓
d.	D.G. sets	✓	✓	✓
e.	Battery room, charger, and DCDB		✓	✓
f.	Pole-mounted or plinth-mounted transformer	✓	✓	✓
3.	Ventilation (Air supply fans/exhaust fans/combination system -as per requirements)	✓	✓	✓
4.	Instruments - Flow, level, pressure, temperature	✓	✓	✓
5.	Internal and outdoor Lighting	✓	✓	✓
6.	Control room	✓ (Common room)	✓	✓
7.	Operator room		✓	✓
8.	Miscellaneous components			
a.	Security guard room	✓	✓	✓
b.	Boundary wall and gate	✓	✓	✓
c.	Parking lots and roads	✓	✓	✓
d.	Storeroom, office, and toilet block	✓	✓	✓
e.	Thrust block	✓	✓	✓
f.	Lifting arrangement in screen chamber, pump floor, silting basin		✓	✓
g.	Internal water supply, sanitary arrangement, wastewater, storm water, and garbage disposal	✓	✓	✓
h.	Material handling equipment {cranes/hoists/gantry as required (at intake, trash rack, and inside pump house)}	✓	✓	✓
9.	Lightning protection to buildings and substations	✓	✓	✓
10.	Aesthetic and environmental considerations	✓	✓	✓

5.2.1 Site and location of pumping station

The site of the pumping station should be on dry land free from flooding risk. In case the site lies in flood prone area, the pumping station should be protected by constructing a proper embankment along with the river and pump chamber and providing adequate drainage arrangement for the pump house and its adjoining area. The pump/motor floor shall always be kept 1-2 m safety margin above high flood level (HFL) with due consideration of flood risk and should remain approachable by a vehicle even in peak monsoon. The site should have sufficient area to locate all the components of the pumping plant as mentioned and preferably on even ground and adequately above HFL. The tapping from the power grid of the supplier should preferably be as near as possible to the pumping station consistent with the reliability of supply, to avoid the high-cost involvement in obtaining power supply from a distant grid. The pumping station should have easy access for heavy vehicles carrying machines, hoisting equipment, etc. minimum of 3 m clear width (excluding pipe, pipe collar, railing, flowmeter, lighting stand post, cable tray, thrust block/wall, etc.) shall be available in approach road/bridge. Sufficient spaces should be provided for transformer substation, water hammer control device, service roads, parking lots, loading areas, heavy lifting equipment, roadside warning signals, stores, security, toilets, etc.

5.2.2 Dedicated Independent Electric Feeder

In the case of all water works and pumping stations, it is preferable to insist on a dedicated independent electric feeder, as these installations are in operation round the clock, throughout the year. Electric substation is required if the power load is 63/100 kVA or higher. The definition given by Electricity Supplying Authorities regarding independent feeders is given below.

An 'independent feeder' would be a feeder in which electricity is supplied only to a single consumer at his own cost relying upon the words "to only that consumer".

Wherever independent electric feeder is not available, diesel generator shall be provided as standby power supply.

5.2.3 Inlet Channel for Intake

The inlet channel to the settling tank shall receive water through the outlet conduit emanating from the intake structure. A minimum velocity of 0.8 m/s should be maintained in the channel. The mechanical bar screen is used to retain debris with a travelling rake mechanism to elevate the floating materials like grass, leaves, etc. along the upstream side of the bar screen. The bar screen shall consist of steel bars of suitable depth and thickness with generally 15-25 mm clear opening

5.2.4 Trash racks and Screen Chamber

A coarse screen may be installed to remove large matter, like floating wood or stones from raw water. A crane for lifting big obstacles and a lifting device for removing accumulated mud or sand from the basin will be installed. Footsteps will be provided for the descent into the basin.

The trash racks may be classified into the following types by their constructional features and the methods of installation:

(i) Type 1 - Removable section racks which are installed by lowering the sections between side guides or grooves provided in the trash rack structure so that the sections may be readily removed by lifting them from guides. These are generally side-bearing types.

(ii) Type 2 - Removable section racks in which the individual sections are not installed between guides in the trash rack structure but are placed adjacent to each other laterally and in an inclined plane to obtain the desired area of flow. Since rack sections may easily be displaced, these have to be secured in place with bolts located above the water line.

(iii) Type 3 - Trash rack sections that are bolted in place below the water line.

Other details shall be as per IS 11388.

Inclination in trash racks is provided to take advantage of an increased section of contact. However, trash racks are also installed without inclination in the vertical grooves of the Intake. These may also be split into panels for ease of handling, i.e., raising/lowering by the lifting beam and hoisting structure provided at deck/pump floor level. A self-grappling/un-grappling type of lifting beam mounted on manual/electrically operated chain block hoist is provided at top of hoist structure.

5.2.5 Pre-Settling tank

Pre-settling tanks, which are plain sedimentation tanks, are useful as a preliminary process to reduce heavy sediments preferably before the intake sump. They may be of quiescent or continuous flow.

Factors that influence sedimentation are:

(i) size, shape, and weight of particle;
(ii) viscosity and temperature of water;
(iii) surface overflow;
(iv) surface area;
(v) velocity of flow;
(vi) inlet and outlet arrangement;
(vii) detention periods; and
(viii) effective depth of basins.

The continuous flow type of sedimentation tank is widely adopted. The aspects of continuous flow sedimentation tank hydraulic are as follows:

(i) The velocity of flow of water in sedimentation tanks should be sufficient enough to cause hydraulic subsidence of suspended impurities. It should remain uniform throughout the tank.
(ii) Maximum surface loading of 60 $m^3/day/m^2$ and a hydraulic retention time (HRT) of three to four hours have to be provided.
(iii) Two settling tanks, one working and one stand bye should be provided in case of quiescent flow.

Refer to Section 8.2 of Part A of this Manual for further details.

5.2.6 Raw Water intake and sump (raw and clear water)

Raw water intake (popularly also called jack well) is designed keeping in view the period of minimum inflow level, so that, the inlets of the suction pipes or bell mouths of pumps as per pump selection always remains submerged with adequate submergence. Please refer section 5.2.7 for details.

Normal practice for all small, medium, and large water supply systems is to design an intake for at least 1.5 times the design flowrate in ultimate stage. Balancing capacity is not an applicable parameter for raw water intake as the inflow rate from the source always matches the outflow/pumping rate. Shape of intake may be circular for small scheme and circular or rectangular for medium scheme. Intake for large scheme shall preferably be rectangular.

Adequate balancing capacity in the raw water/clear water sump is required to overcome variation in discharges of raw water pumps and clear water pumps due to ± tolerances in discharge as per IS and/or substantial increase in discharge of raw water pumps due to lower head consequent to higher water level at source. The balancing capacity of sump shall be referred from Table 2.7 in Chapter 2 Part A. The sump in small and medium scheme may be circular. In case of large scheme, rectangular sump is preferable. Water depth in sump shall be 3-4 metres. Pump/motor floor level of intake and sump shall be at least 0.75-1 m above surrounding/finished ground level or 1-2 m above HFL; whichever is higher.

Spaces for number of working and standby pumps in ultimate stage shall be planned even though, initially, the number of pumps installed shall be as per planning for immediate/intermediate stage. As regards to intake for large scheme, wherever possible, it is advisable to keep space for additional one pump for contingency during life of intake of 50 years as the construction of a new intake is costly and time-consuming.

5.2.7 Intake/Sump Design

5.2.7.1 The objectives of intake/sump design

Detailed consideration needs to be devoted to the intake design to serve various objectives in dry-pit as well as wet pit as follows which are based on IS 1710 and international standards:

(i) to prevent vortex formation;
(ii) to obtain uniform distribution of the inflow to all the operating pumps and to prevent starvation of any pump;
(iii) to maintain sufficient depth of water to avoid air entry during drawdown.

5.2.7.2 Guidelines for Intake/Sump design

Figure 5.1 below illustrates the recommended and the not-recommended practices for sump or intake design. The following points are to be noted in this respect.

(i) Avoid mutual interference between two adjoining pumps by maintaining sufficient clearance, the dimension 'S' in Figure 5.1 is equal to 2 D to 2.5 D.
(ii) Avoid dead spots by keeping rear clearance, dimension B to a maximum of 0.75 D from the centre line of the pump inlets/bell mouths. A dummy wall may be provided, if necessary, in a clear water sump. The top of the dummy wall shall be a minimum up to low water level (LWL). A dummy wall for rear clearance is not advisable in intake which obstructs silt removal. A cone underneath the bell mouth is an adaptable solution to prevent vortex problems.
(iii) It is not advisable to provide dividing walls/baffles in raw water intake which obstructs silt removal. In the case of a clear water sump, dividing walls may be provided between the adjacent bell mouths ensuring that the front edges of bell mouths and the dividing walls are in line and the ends of dividing walls are ogive.
(iv) Provide tapered walls between the approach channel and the sump. By this, the velocity should reduce gradually to about 0.3 m/s near the pumps. This also helps to avoid sudden changes in the direction of the flow. The angle of tapered walls shall be a maximum of 10 degrees.
(v) Avoid dead spots under the suction bell mouth by maintaining the bottom clearance, dimension 'C' between D/4 to D/2, preferably D/3 as shown in Fig. 5.2. It is important that dimension 'C' should NOT be less than D/4; otherwise, peripheral approach velocity shall be higher than inlet velocity at bell mouth which can cause flow disturbance at the inlet to bell mouth. It is to be noted that in the case of raw water intake, it is not practicable to adhere to dimension 'C' allowable maximum up to D/2 as a margin for silt accumulation of about 500-1,000 mm is required. Thus, actual 'C' is excessively higher and shall create vortex disturbance. As a remedial/preventive measure, a Cone or Concrete/Metallic Splitter underneath the bell mouth is necessary and shall be provided, preferably during construction of intake and raw water sump.
(vi) Either splitter or cone shall be provided if a vortex problem occurs as shown in Fig. 5.4 as corrective measure. A splitter or cone is not necessary if 'C' is between D/4 to D/2.

(vii) Avoid sudden drops between the approach channel and the pump well/pump pit in intake and sump. A slope of a maximum of 10° is recommended as shown in Fig. 5.2 so as to achieve adequate water depth for submergence parameter. A suction pit as alternative to floor slope is not advisable for water supply system as this causes waterfall effect and unacceptable flow disturbance. (Such suction pit with steep slopes/haunches on sides to prevent deposition of solids, can be, however accepted for sewage pumping system)

(viii) The floor in the approach bay to the pump suction should be flat up to at least 5D.

(ix) V, the velocity of flow in the pump pit, when water is at LWL, shall not exceed 0.3 m/s.

(x) No cross flow greater than 0.5 V is allowed in the pump pit.

(xi) Within 5D on the upstream side from the centre of suction/bell mouth, if any pier/column is positioned, its sides should be rounded off and downstream sides should be tapered. As far as possible, the approaching flow should directly pass to the pumps without any swirl, change in flow direction and without any obstruction in the flow path.

(xii) Follow-up action shall be taken if dimensions and parameters for vortex-free operation are not fulfilled. The recommended actions for large and important pumping stations are either, or both, as follows:

a. Computational Fluid Dynamics (CFD) Analysis should be carried out for medium and large pumping station. Refer to **Annexure 5.2**.
b. Sump model test should be conducted for large pumping station. Refer to **Annexure 5.2**. Remedial measures concluded after CFD analysis and/or sump model test shall be implemented.

(xiii) For small and medium pumping stations, one of the methods indicated in Figure 5.4, as per applicability, can be adopted to eliminate vortex problems in pump pits.

(xiv) Circular sump and pump house

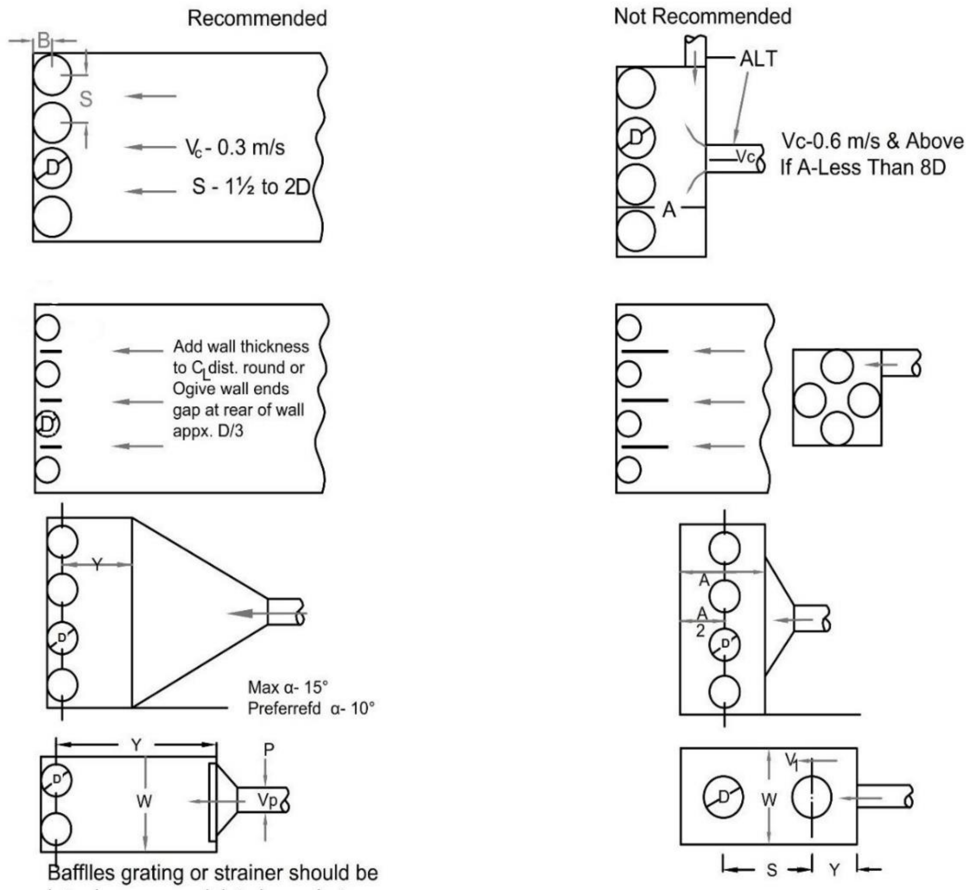
Circular sumps are very popular in India as they are economical in terms of construction costs, easy to construct, and offer compact layout. Figure 5.3 (b) and 5.3 (c) shows typical circular sumps for two pumps and three pumps respectively located at centrelines. Important design dimensions and aspects are as follows:

a. Floor clearance (C) between lip of bell mouth and the bottom for clear water sump shall be D/2 where D = diameter of suction bell mouth. In raw water sump, the clearance shall be based on silt margin.
b. Centre to centre spacing between adjoining bell mouths shall be 1.5 D ensuring that the clearance (C_b) between adjoining bell mouths shall not be less than 100 mm or clear gap, i.e., working clearance of minimum 500 mm between two adjoining pumps and motors, whichever is higher.
c. Wall clearance (C_w) shall not be less than D/4 subject to minimum of 100 mm or wall clearance of minimum 400 mm from motor, whichever is higher.
d. The submergence (S_b) above the lip of bell mouth shall be worked as per guideline in (xvi) below.
e. The diameter of sump shall be worked out fulfilling the dimensions stated in ii and iii above.
f. The inflowing pipe shall be at an elevation with partly or fully below LWL to avoid air entrainment and disturbance due to cascading of flow.

(xv) Sump model tests are required to be carried out if the pumping station falls in the following categories:

a. Non-uniform or non-symmetric approach flow to the pump sum exits (e.g., intake from a significant cross flow, use of dual flow or drum screens, or a short radius pipe bend near the pump suction, etc.)
b. Flow greater than 2.52 m^3/s per pump or 6.31 m^3/s per station

c. Circular sump pumps with discharge greater than 0.315 m^3/s



W/Y	1.0	1.5	2.5	4.0	10.0
Y_m	3 D	6 D	7 D	10 D	15 D
Vpm	0-3	0-6	1.2	1.8	2.5

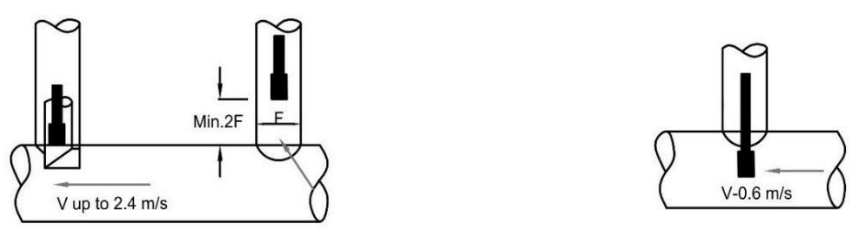


Figure 5.1: Multiple Pump Pit

(xvi) Submergence Sb is to be worked out on the basis of the Froude number, FD using the following two equations.

$$F_D = \frac{V}{\sqrt{gD}}$$

$$Sb = D(1 + 2.3F_D)$$

Where

V = flow velocity, m/s

G = acceleration due to gravity, 9.81 m/s^2

D = bell mouth outside diameter, m

Sb = submergence above the lip of bell mouth

Therefore,

H = Sb + C (actual clearance)

Where H is the minimum depth of water required above the bottom of the sump and C is actual bottom clearance under bell mouth.

Keep adequate submergence of the pump under the LWL as per the dimension H to prevent the entry of air during drawdown and to satisfy NPSHr.

(xvii) Position of trash - rack dimension 'A' is minimum 5D. (Dimension A, however, usually exceeds 5D as Y is also equal to 5D.)
Note: Dimension 'D' is the outside diameter of the suction bell mouth at the inlet which can be derived for dimensions of parameters and hydraulic design of pump bay for vortex-free flow conditions by calculating inside diameter by keeping inlet velocity 1.2 to 1.4 m/s and adding thickness to it.

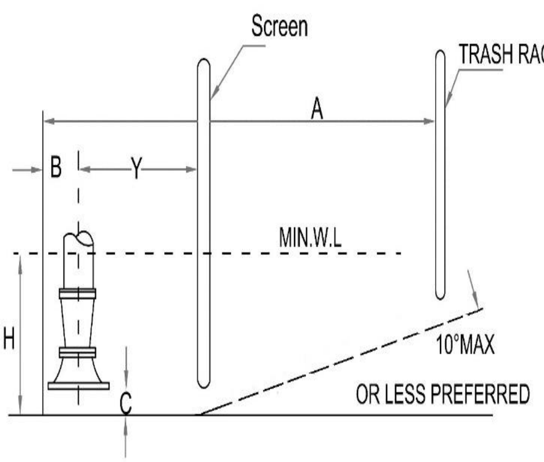


Figure 5.2: Sump dimension elevation view

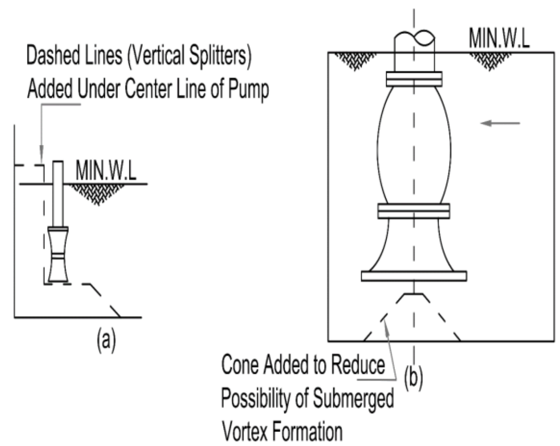


Figure 5.3(a): Vertical Splitters/Cone in the Sump

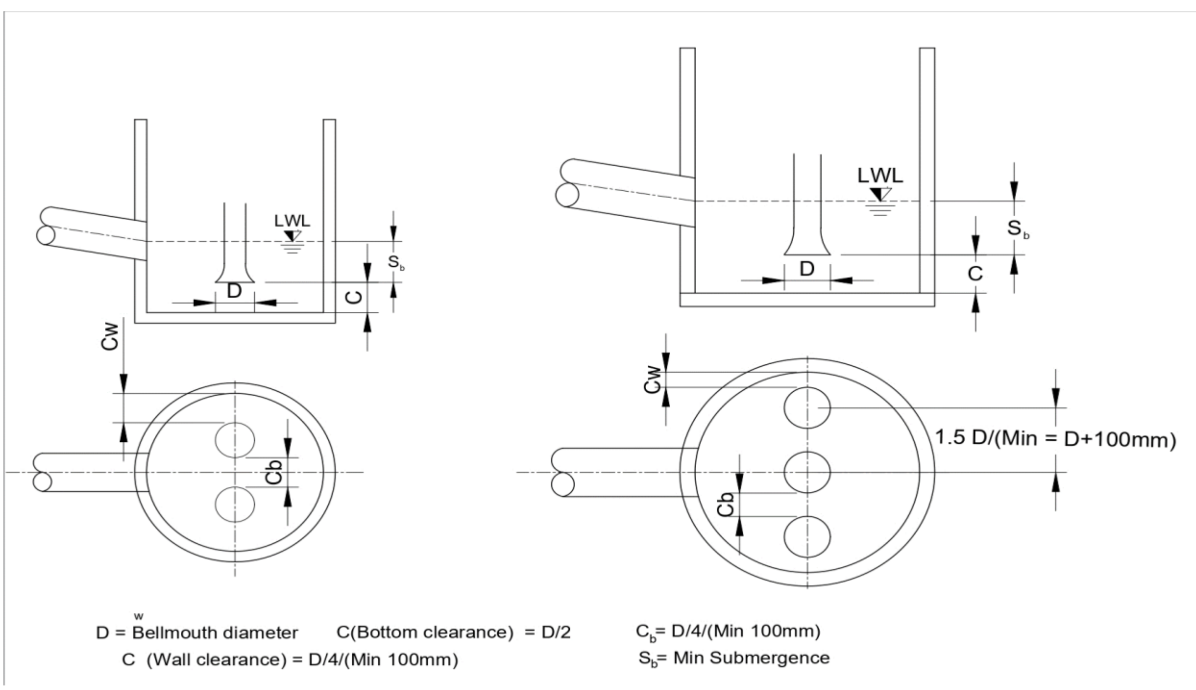


Figure 5.3(b): Two Pumps in Circular Sump **Figure 5.3(c): Three Pumps in Circular Sump**

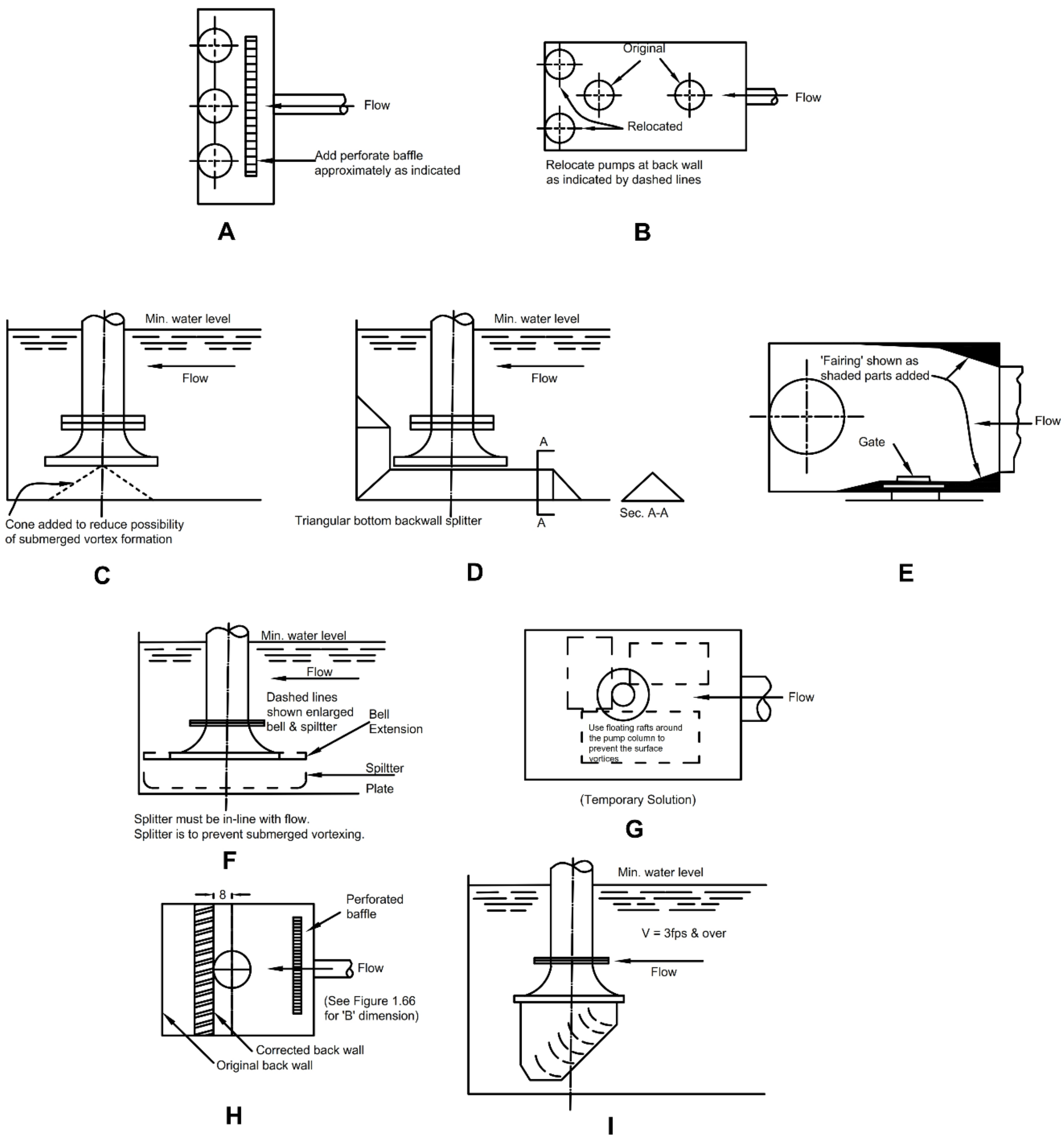


Figure 5.4: Common Methods for Eliminating Vortex in Sumps

Source: Hydraulic Institute ANSI/HI 2000 Edition Pump Standards

5.2.7.3 Piping Intake from Dam

In some impounding reservoirs, where raw water is to supply on downstream of the dam, a pipe outlet is provided from the dam. In such a case, the outlet pipe is extended up to a suitable short distance to locate the raw water pumping station. This outlet pipe is used as piping intake for the pumping station by configuring the outlet pipe as suction manifold for installing (a) barrel type VT pumps or (b) centrifugal pumps as discussed hereunder. Refer to Figure 5.5.

(i) Guiding Criterion

The arrangement is also suitable for connecting the suction manifold to individual suction piping of centrifugal pumps. Criteria as under shall be adhered to:

a. Flow velocity in inlet pipe/suction manifold shall not exceed 2.4 m/s.
b. Velocity in the annular area between barrel and VT pump shall not exceed 1.5 m/s.
c. A 90° long radius bend shall be provided between individual suction/inlet pipe and barrel. Tip of suction bell mouth shall be above the upper tip of 90° bend.
d. LWL in the barrel for VT pump shall be above the lip/tip of bell mouth as per minimum submergence required (based on Froude number) or minimum 1 Db above first stage impeller whichever is higher, where Db is the diameter of the barrel.
e. Velocity in individual suction pipe shall not exceed 1.5 m/s.
f. If individual suction pipes of centrifugal pumps are connected at 90° to the suction manifold. The minimum distance between the individual pump suction nozzle and the centreline of the manifold shall be 8 × Ds where Ds is the individual suction pipe diameter.

(ii) Provision of Surge Control Device on Inlet pipe to Pumping Station

Since inlet pipe/suction manifold is in the continuity of the dam outlet pipe, when all pumps stop on power failure or due to malfunctioning, the flow velocity in the inlet pipe will rapidly decrease causing water hammer overpressure on the inlet pipe and individual suction pipe. A suitable control-free and maintenance-free surge control device is obligatory.

If the elevation difference between FRL in the dam and ground level at the pumping station is less than 25 m, reliable protection can be achieved by providing a MS surge shaft or elevated surge tank as shown in Figure 5.5. The top of the surge shaft/surge tank should be above the FRL of the dam as per the maximum WL rise in the surge shaft/surge tank calculated on basis of numerical analysis for the surge tank/surge shaft. This is necessary to prevent overflow due to WL rise under surge.

Both surge shaft and surge tank are ideal and proven devices requiring no control and no maintenance except for re-painting.

If the elevation difference exceeds 25 m, then the air vessel is the only solution. It may be noted that a surge suppressor/surge anticipation valve is not advisable for raw water application as the pilot valve gets clogged due to impurities and floating material in raw water.

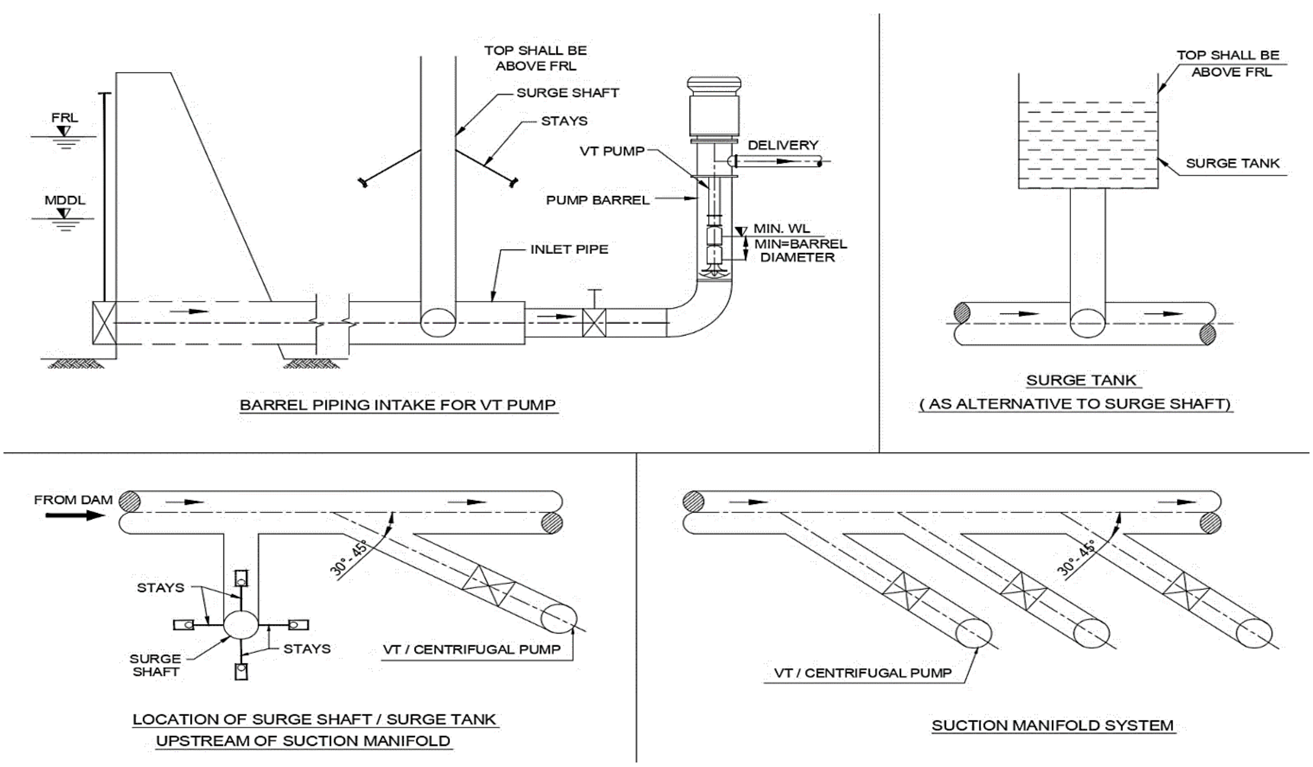


Figure 5.5: Piping Intake, Surge Tank, and Suction Manifold

5.2.8 Pump house

In raw water intake, either vertical turbine pump or alternatively submerged turbine pump can be selected. In case of sump, positive/flooded suction is commonly arranged for the large/medium pumping station by locating generally double suction horizontal split casing pump or end-suction centrifugal pump in adjoining dry pump room such that top of the volute of centrifugal pump is below minimum water level by magnitude of friction loss on suction piping and a small margin for drawdown level and inaccuracies in installation levels. Alternately, submerged centrifugal pump can be used. The general arrangement and dimensions of the pump house are determined by the type and number of working (W) plus standby (S) pump sets to be installed in intermediate stage and additions of pumps for ultimate stage, room for storing spares, Motor Control Centre (MCC) and Programmable Logic Controller (PLC) panel, cable trays, etc. The spacing between adjoining pumps depends on the size of the pump-motor set and working clearance, normally kept at 750-1000 mm or minimum spacing required between bell mouths/suction inlets for vortex-free operation in pump pit, whichever is higher. Sufficient ventilation by providing air supply fans and/or exhaust fans and lighting arrangement should be provided in the pump house. Ventilation for large pumping stations, which is usually done by both air supply fans and exhaust fans. Adequate space should be provided for panel boards, working area for maintenance of pump sets, loading/unloading bay, cable ducts, pump foundation, pipe supports, valve supports, and provision for suction and delivery pipe connections. Lifting equipment shall be provided for the handling of pumps, motors, and other accessories. Pump house should have sufficient headroom to operate the EOT/HOT crane. A minimum of 1.5-2 m clearance should be kept between EOT/HOT and the soffit of the roof beam. Dewatering pumps should also be provided to safeguard against emergency flooding of the below-ground pump houses. The pump house should be designed to maintain the noise levels inside the pump house below permissible limits and to absorb vibrations while pumps are in operation.

1. A ramp or a loading and unloading bay should be provided.
2. The lower floor and upper flower are necessary if the diameter of delivery piping is 350 mm and above and the floors should be so planned that all piping and valves can be laid on the lower floor and the upper floor should permit free movement. The headroom between two floors shall be about 2250-2500 mm.
3. Headroom and material handling tackle.
    a. In the case of a vertical pump with hollow shaft motors, the clearance should be adequate to lift the motor clear off the top face of the discharge head and also carry the motor to the service bay without interference with any other motor/apparatus. The clearance should also be adequate to dismantle and lift the longest column assembly and line shaft.
    b. In the case of horizontal pumps (or vertical pumps with solid shaft motors), the headroom should permit transport of the motor above the other apparatus and motor with adequate clearance.
    c. The mounting level of the lifting tackle should be decided based on the construction and repair of the lifting tackle.
    d. The traverse of the lifting tackle should cover all bays and all apparatus.
    e. The rated capacity of the lifting tackle should be adequate for the maximum weight to be handled at any time.
    f. Depending on the magnitude, duty requirements, capacity, and cost aspects, appropriate lifting equipment from the following alternatives shall be selected.

(i) Tripod and chain pulley block

(ii) Monorail (manually operated)

(iii) Monorail (electrically operated)

(iv) Hand operated travelling crane (HOT crane having three motions, i.e., lifting, travelling, and traversing motions)

(v) Electrically operated travelling crane (EOT Crane having three motions similar for HOT crane)

(vi) Cranes of capacity above 3 tonnes shall preferably be electrically operated.

The lifting equipment (i) is for very small borewell/tube well pumping station and (ii), and (iii) are for a small pumping station and (iv), (v) and (vi) are for medium and large pumping stations.

5.2.9 Suction and delivery pumping system

5.2.9.1 Suction Piping (wherever applicable)

a. The suction piping should be as short and straight as possible.

b. Any bends or elbows should be of a long radius (about four times diameter of suction pipe).

c. As a general rule, the size of the suction pipe should be one or two sizes larger than the nominal suction size of the pump. Alternatively, the suction pipe should be of such size that the velocity shall be about 1.5 m/s. Where bell mouth is used, the inlet of the bell mouth should be of such size that the velocity at the bell mouth shall be about 1.2 to 1.7 m/s.

d. Where suction lift is encountered, no part of the suction pipe should be higher than the highest point in the suction side of the pump body.

e. When a reducer is used, it should be of the eccentric type. Irrespective of positive suction or suction lift, the flat side of the eccentric reducer should be on top.

f. The suction strainer should have a net open area, a minimum equal to three times the area of the suction pipe.

5.2.9.2 Suction Manifold

In the installation, where water is abstracted from a dam by outlet piping or separated sump providing positive suction to centrifugal pumps, a suction manifold is provided with suction branches for individual pumps. Refer to Figure 5.5 under piping intake. Criteria for installation are:

(i) Velocity in manifold shall not exceed 2.4 m/s.
(ii) Velocity in individual suction pipe shall not exceed 1.5 m/s.
(iii) Suction pipes shall, preferably be at an angle of 30-45 degrees to the manifold.
(iv) If suction pipes are laid at 90° to the manifold, the straight length from the centre line of the manifold to the suction nozzle of the pump shall be a minimum of eight times the diameter of the suction pipe.

5.2.9.3 Delivery Piping and Common Header

(i) The size of the discharge piping may be selected one size higher than the nominal delivery size of the pump. Alternatively, the delivery pipe should be of such size that the velocity shall generally be 2.0 m/s; in a large pumping station where the cost of valves is very high, the velocity of 2.25 m/s can be considered.

(ii) Delivery piping connected to a common manifold or header should be connected by a radial tee or by a 30° or 45° bend.

(iii) If more than one pump is required to be operated together, a common header should be designed hydraulically, to reduce the head losses.

5.2.9.4 Dismantling Joint

A dismantling joint must be provided adjacent to the valves both in suction and delivery piping. In the case of delivery branches, the design of the dismantling joint should be such that no pull or moment is transmitted to the pump. Stainless steel bellows can be accepted in place of dismantling joints provided that the tie bolts are adequate to withstand pull under maximum pressure encountered and the shear area is adequate.

Bellow type dismantling joint should not be used in delivery piping as incorporation of this type of dismantling joint causes unbalanced thrust on both ends, i.e., pump end and delivery manifold end.

5.2.9.5 Adequacy of Delivery Piping, Header, and Valves for Water Hammer

Even though a surge protection device is provided for the pumping main, it is advisable the same are designed for protecting the pipeline from common header (excluding) to discharging end point, i.e., WTP/MBR/Sump but not for protecting the delivery piping, header, and valves on pump delivery side. The piping and body of valves should be of proper rating to withstand encountered sub-atmospheric pressure (as applicable) and positive pressure equal to the sum of working pressure plus water hammer pressure in delivery piping without any surge protection devices or shut-off pressure whichever is higher.

5.2.9.6 Valves

(i) Suction Valves

a. When a suction lift is encountered, a foot valve is provided to facilitate priming. The pump can be primed also by a vacuum pump, if the pump is large, usually with a suction pipe larger than DN 300 mm.

   The foot valves are normally available with strainers. The strainer of the foot valve should provide a net area of its openings to be a minimum equal to three times the area of the suction pipe.

b. When there is a positive suction head, a sluice or a butterfly valve is provided on the pump suction, for isolation. The sluice valves should be installed with their axis horizontal to avoid the formation of air pockets in the dome of the sluice valve. In case installation of sluice valve in the horizontal position is not feasible due to space constraints or positioning of electric actuator, sluice valve in a vertical position can be installed.

(ii) Delivery Valves and Reflux valve/Non-Return Valve (NRV)/Dual plate check valve

Near the pump, a non-return (reflux) valve and a delivery valve (sluice or butterfly valve) should be provided. The non-return valve should be between the pump and the delivery valve. The size of the valve should match the size of the piping. A Dual Plate Check valve (DPCV) in place of NRV is acceptable. In an important installation, a manually operated additional sluice valve (SV)/knife gate valve is installed in delivery piping at upstream of the header for attending repairs to the main delivery valve without taking total shutdown.

An electric actuator shall be provided on pump delivery valve if the diameter is 300 mm and above or the pump head is high.

(iii) Isolation valve (IV) and NRV/DPCV on main pipeline Upstream of connecting pipe from Surge Protection Device

One NRV/DPCV along with one isolation valve (SV/BFV) is required to be provided on the main pipeline between the header and the junction point of the connecting pipe from the surge protection device to the pumping main for isolation and improving the effectiveness of surge control device. The surge protection device is designed exclusively for pumping main from common header(excluding) to discharging end.

(iv) Air Valves

Whenever there are distinct high points in the gradient of the pipeline, an air valve should be installed to permit the expulsion of air from the pipeline. If the air is not expelled, it is likely to be compressed by the moving column of water. The compressed air develops high pressures, which can even cause the bursting of the pipeline.

Air valves also permit air to enter the pipeline when the pipeline is being emptied during shut down. If air does not enter during emptying, the pipeline will be subjected to a vacuum inside and the atmospheric pressure outside shall be subjected to undue stresses and, if shell thickness is inadequate, it may collapse. Air valve is also required on downstream of the discharge head elbow for a larger VT pump. One or two air valves are also required on the header. Details on provision and sizing of valves are given in Chapter 11: Pipe and Appurtenances in Part A of this Manual.

An isolation valve (sluice valve) shall be provided for each air valve to facilitate isolation for repairs.

Supports

All valves (including the foot valve, where necessary) and piping should be supported independent of each other and independent of the pump foundation. The supports shall be in RCC construction or fabricated from structural steel or steel plates.

5.2.10 Surge Protection Devices

When starting or stopping a pump (or by operating the regulating valves rapidly) or occurrence of power failure, certain pressure fluctuations are caused, which travel up and down in the pipeline during the transient conditions. This can cause low-pressure zones, particularly at apex points on the pumping main, and subsequently cause very high pressures causing hammer pressures. If such pressure surges exceed the pressure permissible in the pipeline, the pipeline may even burst. To prevent such occurrences, the recommended practices are detailed in section 6.12 and 6.16 of Part A of this Manual.

5.2.11 Electric substation and Substation building

Metering panels that draw power from high tension grids either from overhead or underground cables are installed by the electric supply authority. From metering panels power is fed to the vacuum circuit breaker (VCB) panels and further fed to the transformers to step down to the required operating voltage. Electrical power at operating voltage is fed to the power cum motor control centre (PMCC) panel. PMCC panels, then feed power to various motor control centre (MCC), main lighting distribution board (MLDB), and auxiliary loads. Automatic power factor control (APFC) panels are also installed to improve the power factor of the entire plant. D.G. sets of appropriate kVA should be provided for emergency operations. Spaces for control panels should be planned as per Indian Electricity rules as given below.

I. A clear space of not less than 915 mm in width shall be provided in front of the switchboard (in practice, a front clearance of about 1.4 metres is required so that a person can move in front of the panel even while the servicing work is in progress).

II. In the case of large panels, a draw-out space for the circuit breakers may exceed 915 mm. In such cases, the recommendations of the manufacturers should be followed.

III. If there are any attachments or bare connections at the back of the switchboard, the space, if any, behind the switchboard shall be either less than 230 mm or more than 750 mm in width measured from the farthest part of any attachment or conductor.

IV. There shall be a passageway of minimum 750 mm width from either end of the switchboard clear to a height of 1830 mm.

V. A service bay should be provided in the station with such space that the largest equipment can be accommodated for overhauling and repairs. In a large pumping station housing, more than six or seven pumps, preferably two service bays shall be provided, one at each end or one at one end and another in the middle.

VI. Normally outdoor substation is provided. However, on considerations of public safety and for protection from exposure to environmental pollution, the substation may be indoor.

VII. Following auxiliaries shall be provided:
   i. Lightning arresters.
   ii. Air brake switch/isolator is provided in an outdoor substation. In the indoor substations, circuit breakers are provided. In the case of outdoor substations of capacities 1000 kVA and above, circuit breakers should be provided in addition to air brake switch/isolator.
   iii. Drop out fuses for small outdoor substations.
   iv. Overhead bus bars and insulators.
   v. Transformer.
   vi. Current transformer and potential transformer for power measurement.
   vii. Current transformers and potential transformers for protections in substations of capacity above 1000 kVA.
   viii. Fencing.
   ix. Earthing.

It shall be ensured that the connection for the pumping station is taken from the nearest 11 kV/22 kV/33 kV/66 kV/110 kV/132 kV HV/EHV networks to ensure a 24×7 power supply.

Note: The 11kV/22kV/33kV networks are normally operated with the neutral point earthed through a resistor to limit earth fault current. However, the 11kV/22kV/33kV networks may also be operated with the neutral isolated from the earth during abnormal conditions. The unearthed 11kV/22kV/33kV equipment shall be suitable for continuous operation with an earth fault on one phase and shall be designed to withstand the overvoltage that may occur due to arcing to earth.

5.2.12 Ventilation System

A separate ventilation system with exhaust fans and/or forced ventilation with air supply fans with or without ventilation ducts and/or combination of forced ventilation and exhaust system should be provided. In the motor room, cooling should be provided if required for heat rejection of the motors. The system should be capable of removing heat generated from the motors and panels, to maintain inside temperature within 3 to 5° C above ambient conditions. From the ventilation consideration, a minimum of five to six air changes per hour shall be considered. In case of using self-water cooled submerged/submersible motors, elaborate ventilation in the pump house is not required. The electrical room, MCC, etc. should be ventilated at a rate sufficient to provide five to six complete air changes per hour. Ventilation openings should be screened with fine mesh to prevent the entry of birds, rodents, insects, etc.

Heat Dissipation formula

Heat dissipation is one of the deciding factors in designing heat transfer components. We can calculate heat dissipation for cooling air in pumping station heated due to heat generated from motor windings and other miscellaneous items. Losses from motors (Iron loss, copper loss) causes air to be heated. The applicable formula for Q, air flow in m^3/hour is as under:

$$Q = \frac{3.462 x Ks}{t}$$

Where

K_s = heat generated by motors in K_{cal} /hour

t = Permissible temperature rise above outside temperature (generally 3 °C to 5 °C; preferably 5 °C.)

The value of 3.462 cum per hour is for the air flow rate required per Kcal/hour to restrict temperature rise above outside shed temperature by 1 °C.

K_s for motors = kW rating of motor X (1 - motor efficiency) × 860 × M

Where M is number of maximum working motors

It is stated that:

1 kWh = 860 kcal/hour is based on conversions

1 kWh = 3.60 × 10^6 joules (ii) 1 K_{cal} = 4 .19 × 10^3 joules

Hence, 1 kWh = 860 Kcal/hour (rounded)

5.2.13 Lighting

The interior of the pump house should be provided with a sufficient lighting system specially designed to achieve the best illumination suited to the station layout. Energy efficient fluorescent fixtures are preferred. Lighting should be at adequate illumination levels. For routine service, inspections and maintenance activities are as given in Table 5.2.

Table 5.2: Lighting

S. No.	Area	Illumination level in Lux
1	Substation building	200
2	Pump House	200
3	Control room	250/300
4	Transformer room, D.G. set, etc.	200
5	All other indoor areas	100
6	Outdoor plant area	20
7	Roads	10

(Source: IS 3646 and IS SP72 National Lighting code)

5.2.14 Control Room

The control room for the large pumping station should be equipped with supervisory control and data acquisition (SCADA) control system and be provided with air conditioner. One number of PLC should be installed in the control room with necessary equipment and switches for operation as required. SCADA system will be comprised with the indication of level in the settling tank, sump, flow of raw water, and turbidity, and pH of raw water.

5.2.15 Operator Room

The officer in charge of the plant sits in this office and keeps a watch on all the activities of the plant for its satisfactory functioning. He maintains the record of workers and employees, their remuneration and salaries, spare parts for operation and maintenance, their proper consumption, etc. A water testing laboratory should be provided for all large and medium waterworks, as described in the section 7.7 in Part A of this manual. A telephone should be provided for better control and management of waterworks.

5.2.16 Transformer and Electrical Installation

A supply grid network is generally available in towns and cities for the distribution of power. The elevated tanks are commonly located in such areas. Therefore, it would be economical preferably to opt for transformer substation. Power supply connection to the transformer substation or the pump house can be obtained from the power supply authority after payment of the estimated cost, including additional fees as admissible under their company rules and regulations. Panel spacing and layout in the pump house should be in accordance with Indian electricity rules as described in the preceding section on the large pumping stations.

5.2.17 Miscellaneous Components

Security guard room should be located at the entrance and exit of the plant premise. It serves as a checkpoint to monitor and maintain control over men or vehicles entering and leaving the plant premises. Necessary amenities should be provided for the guards.

The plant area should have a boundary wall all around the premises 1 to1.6 m high above ground level preferably having two layers of barbed fencing over top of the wall. Steel gates should be provided wide enough to permit heavy vehicles, cranes, etc.

Proper lighting arrangements should be made for the whole waterworks campus area. Parking lots for large pumping stations are commonly prescribed for five number of light vehicles, heavy vehicles, tall trucks, big cranes, etc. Wide roads for easy and comfortable movements of these vehicles should be provided inside the plant premises.

Proper arrangements for water supply and sanitary installation within the plant should be made with satisfactory disposal of wastewater to a nearby sewerage system. The storm water drainage system for the site shall be provided and all overflows from the plant shall be laid to storm water drainage system. Plant premises should be maintained neat and clean by proper garbage disposal. Dewatering pumps shall also be provided to remove unwanted water that may accumulate due to some leakage from the pump floor.

i. Aesthetic consideration

Typical low-cost measures to enhance visual quality should be employed:

- allowing adequate area of natural and planted vegetation;
- enclosing unsightly objects such as storage tanks, etc.;
- using local building materials that blend in with the surrounding architecture;

- providing underground utilities (power supply, phone lines, etc.).

ii. Environmental consideration

a. Air Quality

Diesel generators or engine-driven pumps are potential air quality polluters that may be replaced by natural gas or purely grid-supplied electrical energy.

b. Noise

Noise attenuation is a necessary concern near residential areas. Noise level shall not exceed 85 dB measured at a 1.2 m distance from the pump-motor set and vibration level for pumps shall conform to the provisions given in IS 14817 (Part 3) or ISO 10816. Wherever practicable, one or more of the following measures may be adopted:

- Use submersible pump.
- Where submersible pumps are not practicable, use an electrically driven motor. If an engine is used, provide mufflers.
- Build pump house from concrete or masonry.
- Sound insulation of the pump house wall may be an option.

iii. Other considerations for a specific situation

a. Cooling water system (in case of Closed Air Circuit, Water (CACW) motors and for bearings)

- CACW coolers are excellent for cooling generators and large electrical systems, no matter the environment.
- It circulates the water at a temperature lower than the ambient temperature through an element that cools a generator or motor.

b. Forced water lubricated pumps

- When the pumping media (raw water) is hazardous, dirty, and contains solid and abrasive contents, not suitable for bearings, a forced water lubrication system should be used.
- Before deciding on the feasibility of the system, the pump manufacturer should be consulted with a detailed water chemical analysis report. The pump manufacturer will supply a schematic for the forced water system, as well as the amount of water needed for shaft tube and thrust bearing cooling per pump.
- The time required to start the booster pump before starting the main pumps will also be provided by the pump manufacture.

c. Water-seal arrangement

- For effective operation, many pumping arrangements (like VT, horizontal centrifugal, etc.), including those with packing seals and mechanical seals, rely on seal water. Seal water serves three functions: cooling the seal and shaft, lubricating the seal, and flushing impurities from the system.

d. Vacuum priming pumps

- The centrifugal pumps are unable to pump air, which means that when the pumps are taken off-line for maintenance or some other reason, they need to be completely filled with liquid again for expelling air from the pump before they will operate properly.
- The vacuum priming system is used to initially pump out air from the pump which causes drawl of water from suction sump into the pump. When the pump casing is full of water and water stream coming out from vacuum pump is without any air, priming is considered complete.

5.3 Small pumping station

The small pumping station is built either to fill drinking water in elevated reservoirs for distribution in its command area or to boost water in the certain low-pressure zone of the project area. Components of the small pumping station are listed below:

a) Site and location
b) Suction sump
c) Pump house
d) Pole-mounted transformer or transformer room
e) Ventilation and lighting
f) Water supply, toilet facilities, roads, etc.
g) Aesthetic and environmental considerations

a. Site and location

Pumping stations for filling water into elevated tanks are generally located within premises of elevated tanks preferably when the supply main is not far away. In cases where a suitable site and required land area are not available in densely populated towns and cities, it would be expedient to lay a small branch pipe up to the premises of the elevated tank for the construction of a small pumping station to fill the tank.

b. Suction sump

A suction sump of reinforced concrete should be constructed either circular or rectangular having a balancing capacity of minimum 1.5 hours at the discharge rate of the pump. The top of the sump should be covered with an RCC slab with a manhole of 500 mm diameter having an RCC or W.I. manhole cover. The sump top should be at least 500 mm above ground level.

c. Pump house

A pump house should be constructed in RCC or masonry near the sump keeping the long wall of the pump house parallel to the long wall of the sump. The pump house should have adequate space for 1 (W) and 1 (S), each rated for 100% flowrate or alternatively 2 (W) and 1 (S) with each pump designed for 50% flowrate and empty spaces structurally and hydraulically designed for additional W + S pump sets for ultimate stage, electrical panels, and sufficient working space for operation and maintenance of pump sets and allied equipment. A hand operated monorail or electrically operated monorail of adequate capacity shall be provided in the opposite walls 200 mm below the ceiling with a chain pulley block, slings, motor, etc.

(i) Suction and delivery piping (refer to Subsection 5.2.9)
(ii) Transformer and Electrical Installation (refer to Subsection 5.2.16)
(iii) Ventilation and lighting (refer to Subsection 5.2.12)
(iv) Water supply, toilet facilities, and roads (refer to Subsection 5.3)
(v) Aesthetic consideration (refer to Subsection 5.2.17 (i))
(vi) Environmental Consideration (refer to Subsection 5.2.17 (ii))

5.4 Borewell/Tube well pumping station

A borewell pump station is constructed to house pump sets to draw water from the borewell/tube well. Generally, conventional submersible pump with both pump and motor on common single shaft, installed in borewell/tube well below minimum water level is used. Vertical delivery pipe is connected to pump delivery nozzle to top of well above ground level. The delivery piping, valves, etc., are installed in the pump house at ground level.

Sometimes if the well is shallow, a vertical turbine pump is selected. The turbine pump assembly is made up of one or more impellers housed in a single or multistage unit known as a bowl assembly. The impellers are suspended on a vertical line shaft that is housed in a pump column which conducts the water to the surface. The individual sections of the pump column are generally manufactured in 2-3 m length. In the course of lowering the column pipe sections inside the well, they are jointed with threaded couplings or flanged fittings. The pump column is attached at the surface to the discharge head which houses a stuffing box around the shaft and an elbow to divert the discharge of water into the above-ground piping system. Components of this type of pumping station are listed below:

(i) Pump house
(ii) Pumping machinery
(iii) Borewell/tube well
(iv) Pole-mounted transformer or transformer room if load exceeds certain kVA
(v) Delivery piping
(vi) Lighting and ventilation
(vii) Water supply, toilet facilities, roads, etc.
(viii) Aesthetic and environmental considerations

Pump house

The pump house is constructed right over the borewell keeping the borewell in the middle of the pump house. Adequate clear space should be kept around the borewell for the installation of a vertical turbine pump set or submersible pump. Sufficient space for locating the delivery piping, valves, electrical panel, starter, switch, circuit breaker, electrical measuring instruments, etc., should be provided. The ceiling of the pump house should be not less than 5-5.5 metres above the floor of the pump house for lowering and extracting column pipe sections. A hand operated monorail of adequate capacity should be provided with a chain pulley, slings, etc., for lifting the pump, motor, column pipe sections, etc. Alternatively, a tripod with chain pulley block can be used for very small pumping station.

a) Suction and delivery piping (refer to Subsection 5.2.12)
b) Transformer and Electrical Installation (refer Subsection 5.2.19)
c) Ventilation and lighting (refer to Subsection 5.2.15 and 5.2.16)
d) Water supply, toilet facilities, and roads (refer to Subsection 5.3)
e) Aesthetic consideration (refer Subsection 5.2.20 (i))
f) Environmental consideration (refer Subsection 5.2.20 (ii))

5.5 Classes of pumps

All pumps are classified into two major classes:

a. Kinetic energy

- Centrifugal pumps
- Jet pumps

- Airlift pumps

b. Positive displacement

- Rotary pumps
- Peristaltic pumps
- Reciprocating pumps

Of these, the centrifugal pumps and the reciprocating type of positive displacement pumps are more popular. Prominently, the reciprocating pumps are good on high head (high pressure) duties and for metering/dosing requirements. Centrifugal pumps are of mechanically simpler construction and give non-pulsating continuous flow.

The arrow marked on the pump casing is only the direction of rotation and not the direction of flow. The direction of flow has to be found by, i) comparing the suction flange, which is usually larger, than the delivery flange; ii) Pump casing profile.

5.5.1 Pump Types Based on Variable Frequency Drive

A variable frequency drive (VFD) is an electronic controller that adjusts the speed of an electric motor by modulating the power being delivered. Variable frequency drives offer continuous control by matching motor speed to the specific demands of the work being done. However, for intake pumping and clear water pumping, constant speed pumps shall be preferred.

Variable speed pumps are employed when there is a requirement for a change in flow or head due to demand changes over a period. For instance, in a city distribution of water by direct pumping, the terminal head at critical point (at highest elevation node in the operation zone of distribution system) has to be maintained irrespective of demand. During low demand periods, in the case of a constant speed pump, the terminal pressure may become higher as the pump may be discharging lower discharge. In such cases, the pump delivery valves (or the line valves) are throttled to keep the pressure at the required level to avoid excessive pressure. This is detrimental to the pump as it has to work closer to shut-off head, and also results in a waste of power. Alternatively, if the pumps are run at a lower speed and still maintain the end pressure, the pumps will be working close to their best efficiency point (BEP), and near their rated head (at the reduced speed) and thus is a safer option. Selection of speed control option has to be done keeping in view the entire demand range, static head, and other factors. The use of VFD is beneficial where the system is friction dominant. The use of VFDs for most “24×7 Drink from the Tap” systems would be useful.

To understand how speed variation changes the duty point, the pump and system curves are overlaid. Two systems are considered, one with only friction loss and another where the static head is high in relation to the friction head. It will be seen that the benefits are different. In Figure 5.6, reducing speed in the friction loss system moves the intersection point on the system curve along a line of constant efficiency. The operating point of the pump, relative to its BEP, remains constant and the pump continues to operate in its ideal region. The affinity laws are obeyed which means that there is a substantial power reduction obtained together with the reduction in flow and head, making variable speed the ideal control method for systems with friction loss.

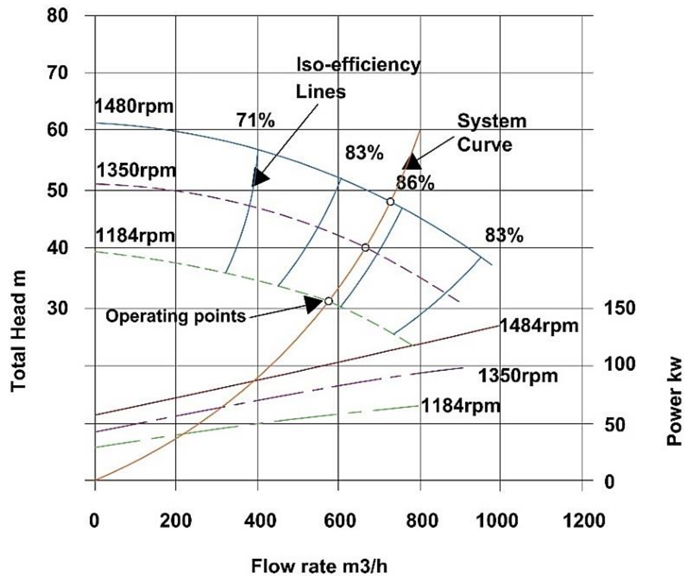


Figure 5.6: Example of the effect of pump speed change in a system with only friction loss

Source: Bureau of Energy Efficiency, "Pumps and Pumping System"

In a system where the static head is high, as illustrated in Figure 5.7, the operating point for the pump moves relative to the lines of constant pump efficiency when the speed is changed. The reduction in flow is no longer proportional to speed. A small turndown in speed could give a big reduction in flow rate and pump efficiency, which could result in the pump operating in a region where it could be damaged if it run for an extended period even at a lower speed. At the lowest speed illustrated (1184 rpm), the pump does not generate sufficient head to pump any liquid into the system, i.e., pump efficiency and flow rate are zero and with energy still being input to the liquid, the pump becomes a water heater and damaging temperatures can quickly be reached.

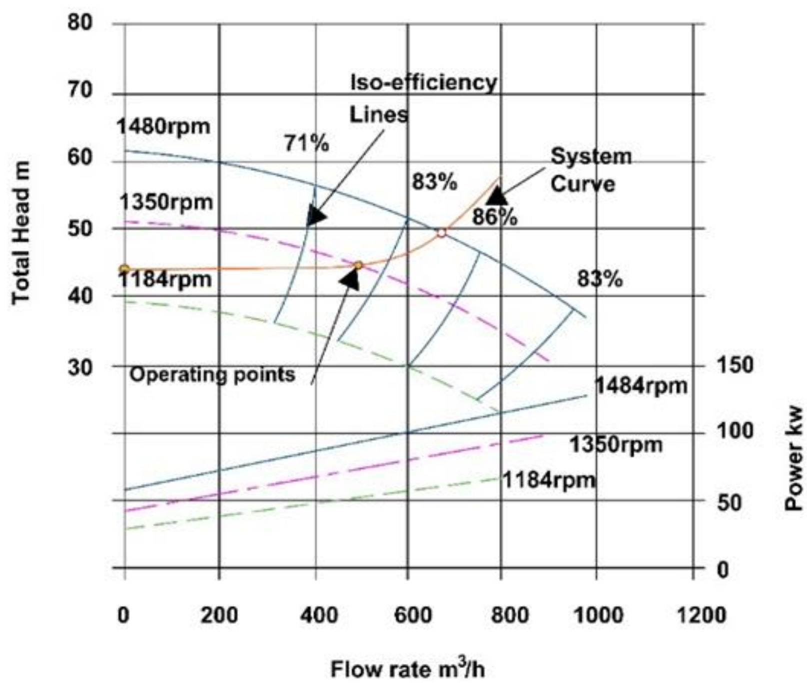


Figure 5.7: Example of the Effect of Pump Speed Change in a System with Static head

Source: Bureau of Energy Efficiency, "Pumps and Pumping System"

The drop in pump efficiency during speed reduction in a system with a static head reduces the economic benefits of variable speed control. There may still be overall benefits, but economics should be examined on a case-by-case basis. Usually, it is advantageous to select the pump such that the system curve intersects the full speed pump curve to the right of best efficiency, in order that the efficiency will first increase as the speed is reduced and then decrease. This can extend the useful range of variable speed operation in a system with a static head. The pump manufacturer should be consulted on the safe operating range of the pump. Further details may be referred at section 5.16.1 of this chapter.

The introduction of VFDs requires additional design and application considerations - additional information can be obtained from pages 12, 13, and 14 of “Variable Speed Pumping - A Guide to Successful Applications” published by Hydraulic Institute Standards, Euro Pump and U.S. Department of Energy.

Motor for VFD system should be VFD Compliant certified by the motor manufacturer.

5.5.2 Pump Types Based on the Method of Coupling the Drive

Some pumps are coupled to the drives, direct through flexible couplings, or are close-coupled or are distantly driven through belt and pulley arrangement, sometimes with gearing arrangement or even with variable speed arrangement.

5.5.3 Pump Types Based on the Position of the Pump Axis

Pumps normally work with their axis horizontal. Vertical turbine pumps, borewell submersible pumps and volute type sump pumps have their axis vertical. Dry-pit pumps are often arranged to work with their axis horizontal.

5.5.4 Pumps of Types Based on Constructional Features

For the purpose of maintenance, pumps are made with axially split casing or with a back pull-out arrangement. Pumps for high heads are built with multi-staging. Pumps to handle solids and sewage are provided with access hand holes for inspection and cleaning the choking and also with the provision for flushing and draining. Submersible pumps to handle raw water should be with mechanical seals. In this manner, a large variety of constructional features are provided in pumps for different purposes in different situations.

Pumps are also made in a variety of materials, to withstand corrosion, erosion, abrasion, and for longer life under wear and tear.

5.6 Design Features of Centrifugal Pumps, Vertical turbines, and Submersible Pumps

5.6.1 Design Types of Pumps

The type of design is as given below.

a) Two types based on the type of casing
   - Turbine (diffuser)
   - Volute

b) Three types of designs based on the flow profile of impellers
   - Radial flow
   - Mixed flow
   - Axial flow

   Casing for above three types may be volute or diffuser

c) Three types based on dry or wet pit installations

Centrifugal pump	Both pump and motor dry
Vertical turbine pump	Pump in a wet pit (submerged) and motor dry
Submersible pump/Submerged turbine pump/Submerged centrifugal pump	Both pump and motor in a wet pit (submerged)

5.6.2 Features and Suitability of Various Types of Pumps

This subsection describes features and suitability of various types of pumps

5.6.2.1 Turbine pump

In a turbine pump, the impeller is surrounded by diffuser vanes that provide gradually enlarging passages in which the velocity of water leaving the impeller is reduced, thereby, converting kinetic energy into pressure energy and thus, develops pump head.

VT pump under (5.6.2.6) below and conventional submersible pump under (5.6.2.7) below are examples of turbine pumps.

5.6.2.2 Volute pump

The volute pump differs from the turbine pump in that there are no diffuser vanes, and the impeller is housed in a spiral-shaped case. The velocity of water is reduced upon leaving the impeller, thus transforming velocity to pressure head.

The choice between turbine and volute pumps depends on the condition of use. Ordinarily, the volute design is preferred for large capacity, low/medium head applications whereas turbine design is desirable where high heads are involved.

Centrifugal pump is normally with volute.

5.6.2.3 Radial flow pumps

In radial flow pumps, pressure is developed by centrifugal force. The water normally enters the impeller hub axially and flows radial to the periphery; the impellers may be single or double suction. The impeller may have either straight or double curvature and the pump shaft may be horizontal or vertical.

5.6.2.4 Mixed flow pumps

In mixed flow pumps, the liquid/water enters axially and discharges in partly off radial direction. The head is created by centrifugal force and a lift of the vanes on the water. The casing can be volute or diffuser type. The pump is either single or double volute and may be either single or multistage. These pumps are applicable for medium head application.

5.6.2.5 Axial flow pumps

Axial pumps are also known as propeller pumps and develop the head by the lifting or propelling action of the vanes on the water/liquid. They have a single inlet impeller with flow axially and discharging axially. These pumps are commonly used for large flows and very low head installations such as lift irrigation schemes.

5.6.2.6 Vertical Turbine (VT) pumps

The impellers are mounted on impeller shaft along with diffusers housed in bowl assembly. The impeller shaft is connected to vertical line shaft that is housed in a column assembly which conveys the pumped water to the surface. Individual sections of the column assembly are generally manufactured in 1.5 to 3.0 m length. Generally, column assembly in 1.5 m length which results in practically true rotation of line shaft, reduces critical speed, and also reduces height of installation level of crane, is advisable for all VT pumps, and is essential if length of column assembly exceeds 12 m. In the course of lowering the column pipe sections inside the well, they are usually with flanged couplings. The top column pipe is attached at the surface to the discharge head which houses a stuffing box around the shaft and an elbow to divert the discharge of water into the above-ground piping system. The impeller shaft is of high tensile steel or stainless steel (SS). The line shafts are of SS for column assembly of any length and essential if length exceeds 12 m, as the diameter of SS shaft being higher, is beneficial for reducing critical speed and true running of line shaft.

The VT pump may be radial flow type as per (5.6.2.3) above or mixed flow type as per (5.6.2.4) or axial flow type as per (5.6.2.5) above.

A flanged motor is installed above the discharge head. If the motor is hollow shaft (generally applicable for motor up 110 kW), the top line shaft is coupled to hollow shaft of the motor. The thrust bearing is provided in the motor to counter total axial thrust as sum of unbalanced hydraulic thrust in the pump and dead load of rotating assembly of the pump (i.e., impellers, impeller shaft, line shafts, and couplings) and the rotor of the motor.

If the motor is solid shaft, a flexible rubber bush coupling in two halves is provided to couple top line shaft and motor shaft and is located in discharge head. A thrust bearing is housed in the discharge head and is designed to withstand total axial thrust as sum of unbalanced hydraulic thrust of the pump and dead load of rotating assembly of the pump only and, is usually designed for 40,000-50,000 hours of operation, i.e., six to seven years. The thrust bearing in motor counters dead load of the motor rotor.

The discharge head should be mounted on sole plate anchored to foundation. The top of sole plate shall be smooth finished and accurately levelled and permanently anchored in foundation such that levelled sole plate do not need to be disturbed whenever the pump is taken out for repairs. The bottom of discharge head shall also be smooth finished and contact faces of sole plate and discharge head are blue matched.

Three types of lubrication system are used for vertical turbine pumps depending on raw water turbidity:

i. Self-water lubricated (pumped water lubricated)
ii. Oil lubricated
iii. Forced water lubricated

In all cases, the line shaft shall be of non-corrosive material generally stainless steel.

i) Self-Water Lubricated (Pumped Water Lubricated)

Self-water lubricated pumps are the simplest in constructional features as well as maintenance and should be preferred if raw water turbidity is low in river water and impounded reservoirs (dams) where due to settlement in the reservoir, turbidity of pumped water reduces. In many cases, even if peak turbidity during monsoon is up to 500 NTU which lasts for a few days, self-water lubricated pumps are functioning without any significant problem.

The bearings for the line shaft are generally of cut-less rubber and are provided at each flanged joint of column pipes with a bearing holder.

A water slinger is provided to prevent water from creeping into the motor.

ii) Oil Lubricated Pumps

Oil lubricated pumps shall be selected if turbidity is high. The arrangement comprises a shaft enclosing tube for the line shafts. At each joint of the shaft enclosing tubes, a threaded bronze bearing, commonly called line shaft bearing is held at the joint after tightening screw threads. The shaft enclosing tubes are held in position by spiders, one per tube.

Low viscosity oil is passed under gravity at rate of two to three drops per minute through the shaft enclosing tube to lubricate line shaft bearings.

This arrangement requires maintenance to prevent ingress of raw water into the shaft enclosing tube which results in ineffective oil lubrication. Care should be taken to ensure that oil does not leak into pumped water or else it may pose a public health hazard.

iii) Forced Water Lubricated Pump

This arrangement is applicable if turbidity is very high. It is, however, not very common. The construction feature is similar to an oil lubricated pump with a shaft enclosing tube except that line shaft bearings are of cut-less rubber which are located at flanged joints of column pipes.

Pressurised clear water from an external source at pressure higher than pressure of pumped water is passed through the shaft enclosing tube to lubricate line shaft bearings. A water slinger is provided to prevent water from creeping into the motor.

This arrangement requires maintenance to prevent the ingress of raw water into the shaft enclosing tube.

The VT pumps are suitable for following installations.

a. Constructed intake (river/lake/Impounded reservoir)
b. Sump (raw water/clear water)
c. Piping intake from dam wherein the pump can be used as barrel pump as in 5.2.7.3

5.6.2.7 Centrifugal Pump

The centrifugal pump may be radial flow type, mixed flow type, or axial flow type. This type is with volute casing. Depending on number of stages in the pump, the same are classified as a single stage or multistage. Similarly, on the basis of orientation of pumps, they are classified as horizontal centrifugal as pump axis is usually horizontal. If a pump is with vertical axis, the same is classified as vertical centrifugal pump.

Following types of centrifugal pumps are popularly used.

- Horizontal centrifugal end-suction pump: Suction is at end and in horizontal plane. Delivery nozzle is generally vertical.
- Double suction horizontal split casing pump: This type is the most preferred pump as upper half casing can be removed for attending repairs without taking out pump shaft, impeller, etc. Being double suction, net positive suction head required is lower and axial hydraulic thrust is nearly balanced, thus reducing bearing losses and resulting in higher efficiency.
- The centrifugal pumps are suitable for following installations.

  a. Dry well above the sump if suction lift capability is adequate
  b. Dry well by the side of wet well with positive suction by extending suction pipes into wet well
  c. In-line booster pumping station
  d. Piping intake from dam

- Important consideration for deciding floor level for centrifugal pump:

Installation of horizontal centrifugal pump on floor below surrounding ground level to the extent possible should be avoided as in the event of burst of any valve or pipe of individual delivery of pump in the pump house, the motor can be damaged due to water logging on the floor.
A good example of centrifugal pump installations is of Bengaluru water supply systems where the pump mounting floor levels are at or above surrounding ground levels, thus, avoiding such risk. Clear water sumps are at higher ground levels, thus rendering positive suction to the pumps.

5.6.2.8 Submersible pump (conventional)

Submersible pumps have bowl assemblies that are similar to those of vertical turbine pumps. The motor, however, is submerged under water and directly connected to and located just below the bowl assembly. Water enters through an inlet strainer between motor and bowl assembly, passes through the stages, and is discharged to the surface via the vertical delivery pipes. Submersible pumps have become a major type of pump used in domestic wells, and increasing numbers of submersible pumps have been installed in large diameter, high-capacity wells. Submersible pumps have several advantages including the following.

- Motor is easily cooled because of complete submergence.
- Noise level transmitted to ground surface is very low or practically eliminated due to submergence and water column.
- The submersible pump has a hermetically sealed motor close-coupled to the pump. The entire assembly is immersed in the fluid being pumped. The pump is just above the motor, and both of these components are suspended in water. Submersible pumps use enclosed impellers and are easy to install and maintain. These pumps run only on electric power and can be used for pumping water from very deep and crooked wells. Moreover, they are unlikely to be struck by lightning and require a constant flow of water across the motor.
- The submersible pumps are suitable for following installations:
  a. tube well/borewell/dug well;
  b. small intake (if raw water turbidity is low);
  c. sump for small schemes.

Single phase (230 V) and three-phase (415 V) submersible pump-motor sets manufactured in India are as follows:

1 phase: Fractional kW to 2.25 kW	Generally used for a very small rural scheme
3 phase: 0.5 kW onwards	Other schemes

5.6.2.9 Submerged turbine and submerged centrifugal pump sets

Submerged turbine pump and centrifugal pump sets wherein both pump and motor submerged and common shaft provided for pump and motor are manufactured in India and abroad.

The design engineers should arrive at decision after due consideration of merits and demerits. These pumps are, however, very meriting for application where space and time are limited and/or installations where no adequate time is available for construction of civil works. Features of these submerged pump sets, their merits, and demerits including comparison with conventional VT and centrifugal pumps are as follows.

(i) Submerged turbine pump set

This type of pump on detailed consideration of merits and demerits and comparison with conventional VT pump including requirements of civil works may be evaluated as alternative to conventional VT pump. The features of submerged turbine pumps are:

- The pump/bowl assembly is on top and the motor is below under submerged condition; as against the bowl assembly of the conventional VT pump, where it is partly or fully under submerged condition;
- However, in both cases, the column assembly, transmission shaft/line shaft, discharge head, and motor are located above the water level and remain dry.

Figure 5.8 (a) illustrate conventional VT pump. The Figure 5.8(b) shows submerged turbine pump without a can for motor as per present manufacturing practice in India.

- As seen from the figures, the motor of the submerged turbine pump is below the bowl assembly. The pump is without transmission/line shaft and discharge piping is from delivery nozzle of turbine pump.
- In a submerged turbine pump set, the entire axial thrust, comprising the hydraulic thrust in the bowl assembly/pump and the weight of rotating assemblies of the pump and motor, is taken by the thrust bearing in motor as against separate thrust bearing provided in discharge head in the case of a conventional VT pump with dry motor.
- Merits of the submerged turbine pump:
    - No transmission/line shafting, hence eliminating small power loss and maintenance of line shaft bearings.
    - Due to bearings lubricated by grease and not in contact with pumped water, the same pump is suitable for raw water and clear water application.
    - Design of structure is economical as vibration level transmitted to structure is negligible.
    - Noise level is negligible being submerged.
    - No need of elaborate ventilation at operation floor, as the motor which is the major source for heat emission is submerged.
    - Spacing between pump/bell mouth centres can be reduced as motor is submerged and therefore, working clearance is restricted to spacing requirement from aspect of vortex phenomenon.
- Demerits of the submerged turbine pump
    - A common shaft for pump and motor making the entire set out of service even if either of the pump or the motor fails.
    - Motor of turbine pump set is below the pump. Submergence required for vortex-free hydraulic condition is computed above the lip of bell mouth/inlet and generally bottom clearance equal to half of bell mouth diameter is adopted.
    - However, if a submerged turbine pump is chosen, and as a submergence requirement, above pump inlet remains the same, the bottom of the pump well will have to be lowered to accommodate the motor depending on its height. Thus, excess depth of the pump well is required. It therefore follows that if the pump well is designed for a conventional VT pump, a submerged turbine pump set cannot be installed without lowering the bottom of the pump. An important demerit is that if a vortex problem occurs, no remedy is possible in the case of a submerged turbine pump as the motor is near the bottom floor. However, Vortexes can occur with any type of pumps due to poor sump design & hence it is advisable to get sump design checked before installing any type of pump. In a conventional VT pump, remedial measures are always possible as bell mouth is below bowl assembly and near the bottom floor.

- Essential features and improvements required based on the review on international standards, practices, and brochures:
    - A barrel (also called as jacket or shroud) shall be provided enclosing the motor from the bottom of the motor to the pump inlet. The top of the jacket shall be closed, but

not airtight to expel trapped air, if any, in the barrel. Diameter of barrel shall be designed to limit flow velocity in annular space to 1.5 m/s maximum.
  - A well designed and sturdy sole plate arrangement for founding the bend of a vertical discharge piping shall be provided at operating floor. Bottom sole plate shall be levelled and permanently fixed in the foundation. Upper plate shall either be integral with bend or bolted to bottom flange of the bend and shall be fastened to bottom sole plate.

- The pumps are suitable for following installations:

  I. Constructed/Not constructed intake (river/lake/Impounded reservoir)
  II. Sump (raw water/clear water)
  III. Low lying/waterlogged areas at the pumping station prone to floods

(ii) Submerged vertical centrifugal pump rested with auto-coupling

Figure 5.9 (a) illustrates salient features of the pump.

- Merits of the submerged vertical centrifugal pump
  - Regular pump house can be dispensed with, or smaller pump house is required. However, panel room and lifting equipment are required.
  - Width of pump well can be reduced as working clearance between motors for heat dissipation is not required being under water.
  - No need for elaborate ventilation as motor is under submerged condition.
  - Noise level is negligible.
- Demerit
  - A common shaft for pump and motor making entire set out of service even if either pump or motor fail.

- The pumps are suitable for following installations:
  I. Intake
  II. Sump (raw water/clear water)
  III. Low lying/waterlogged areas at the pumping station prone to floods.

(iii) Submerged Horizontal centrifugal pump set with portable base frame and submerged vertical centrifugal pump set with portable base frame.

Figure 5.9 (b) illustrates salient features of horizontal centrifugal pump set. The features of vertical centrifugal pump set are similar with motor on top and pump at bottom with end suction and side delivery.

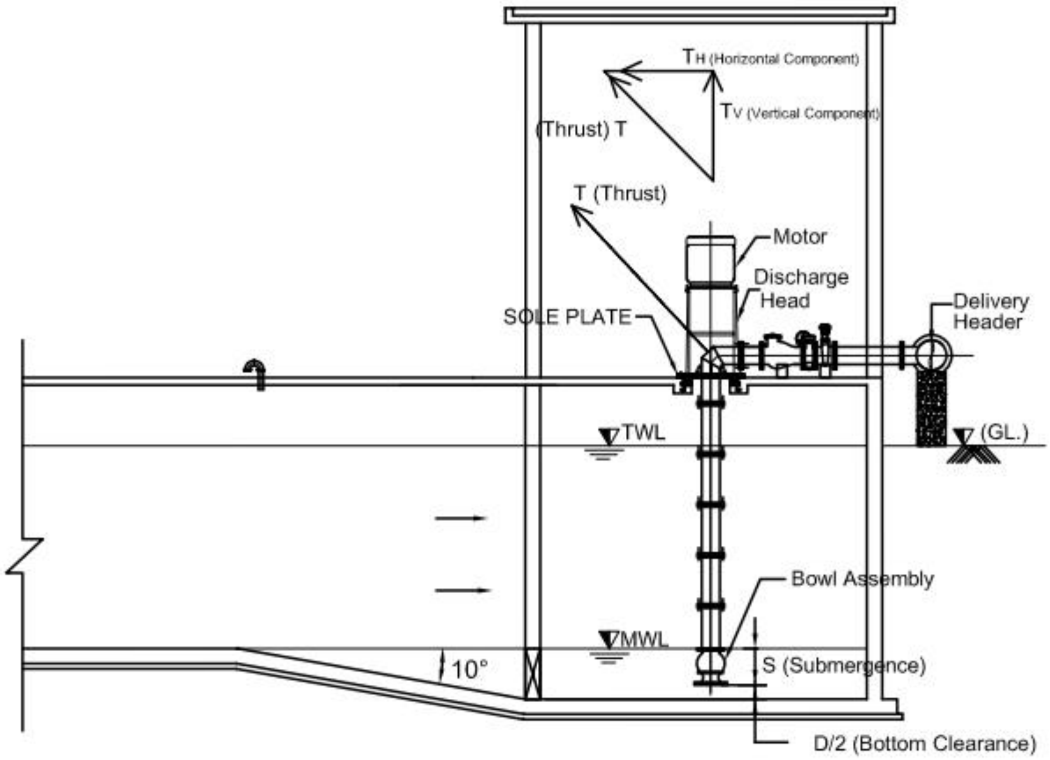
- Merits of the submerged horizontal centrifugal pump

  Merits are same as discussed in (ii) above for submerged centrifugal pump set with auto-coupling.
- Demerits of the submerged horizontal and vertical centrifugal pump:
  - A common shaft for pump and motor making the entire set out of service even if either of the pump or motor fails.
  - Whether a portable base frame simply resting at bottom floor without any anchorage can restrain the pump set under dynamic load during normal running is questionable.
  - Major demerit of installation arrangement of horizontal pump is that the approaching flow passes first to the motor and next to the pump body before reaching to the inlet/suction of the pump. This is contrary to the guideline in the standards for vortex-free design that inflow should approach straight to the suction inlet without swirl or

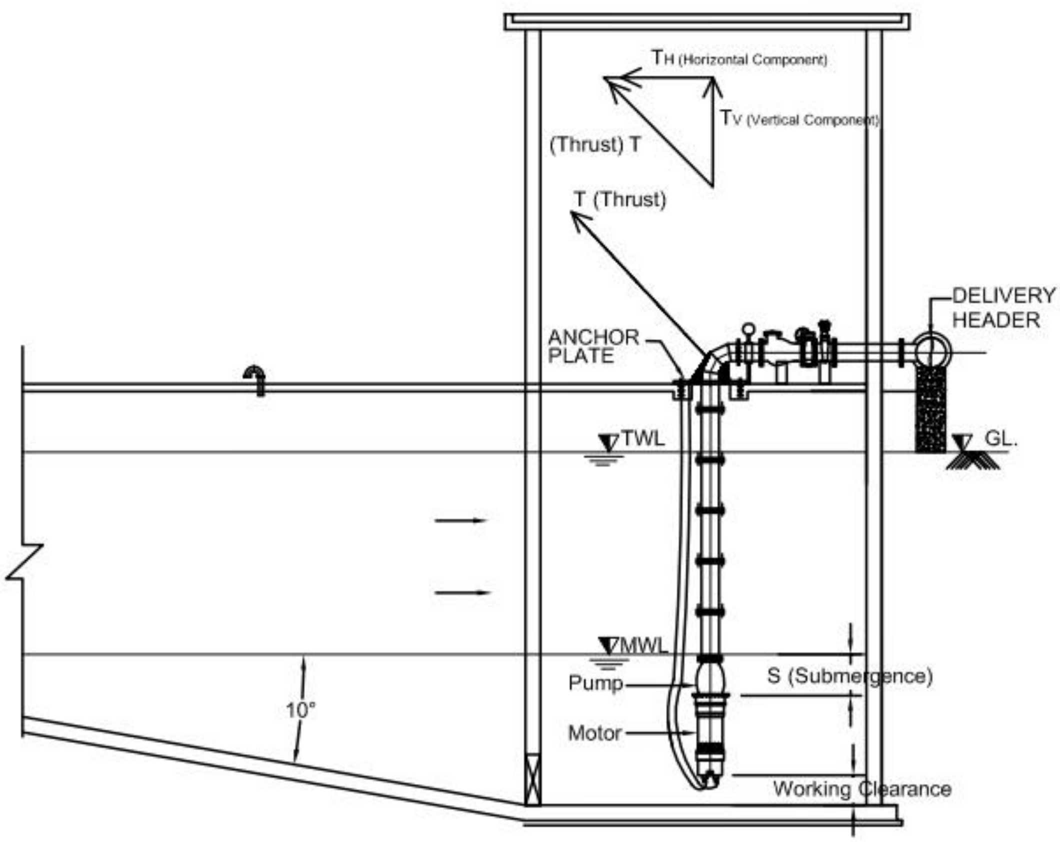
change in flow direction and without disturbance due to any obstruction in flow passage to pumps.
  - Delivery piping from pump delivery nozzle to operation floor is vertical. Hence, when the pump set is taken for repairs, both the pump set and the vertical piping need to be lifted up. Lifting of such eccentric load can be very cumbersome.
  - For horizontal centrifugal pump set due to being end suction and without suction piping, no remedial measures for vortex prevention can be adopted. Also, it is not possible to maintain bottom suction clearance equal to half of the suction bell/inlet diameter. The portable frame may also cause flow disturbances at the bottom.
  - For vertical centrifugal pump set, the portable frame may cause flow disturbance at bottom.

- Features required for betterment for installation in sump:
  - Orientation of the pump-motor set should be changed such that approaching flow directly passes to the pump.
  - It is advisable to provide proper rigid foundation for the pump-motor set at bottom level subject to feasibility.
  - Frame of vertical centrifugal pump shall be improvised such that the front part of the frame does not cause or minimise obstruction in the flow path to the pump suction.

- The pumps are suitable for following installations:
  I. Constructed/Unconstructed intake
  II. Sump (raw water/clear water)
  III. Low lying/Waterlogged areas at the pumping station prone to floods

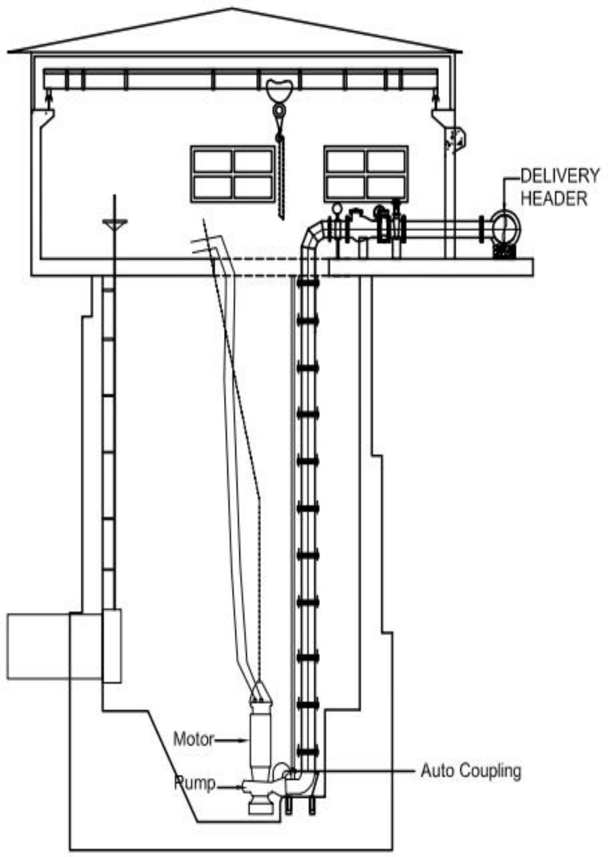


(a) - Conventional VT Pump Installation

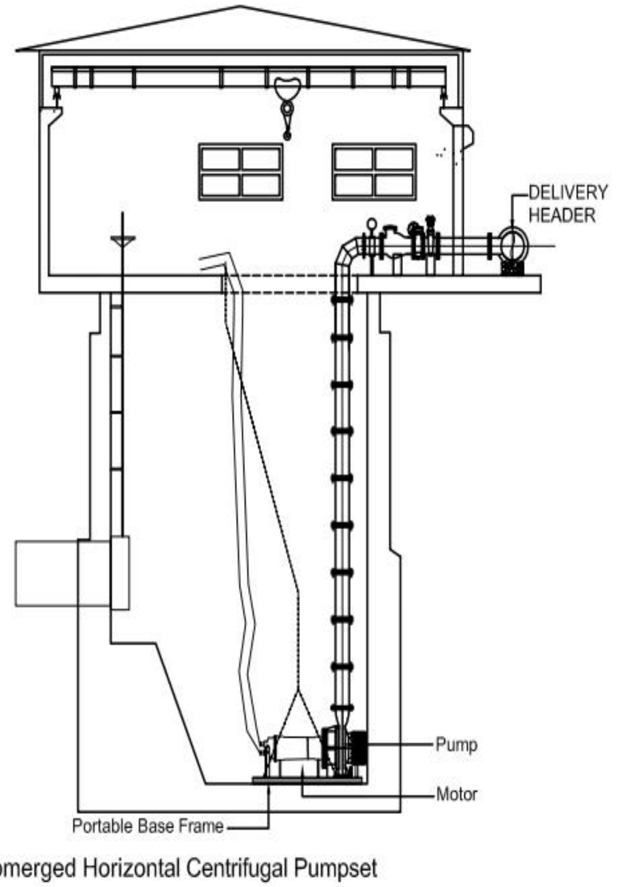


(b) - Submerged Turbine Pump Installation

Figure 5.8: Relative Installation Arrangements of (a) Conventional VT Pump set and (b) Submerged VT Pump sets



(a) Submerged Vertical Centrifugal Pumpset with Auto Coupling



(b) Submerged Horizontal Centrifugal Pumpset

Figure 5.9: Installations (a) Submerged Vertical Centrifugal with auto-coupling and (b) Submerged Horizontal Centrifugal Pump set with Portable Base Frame

5.7 Criteria for Pump Selection

Prior to the selection of a pump for a pumping station, detailed consideration has to be given to various aspects, viz.:

a. Nature of liquid may be chemicals or if water, then whether raw or treated
b. Type or duty required, i.e., whether continuous, intermittent, or cyclic
c. Present and projected demand and pattern of change in demand
d. The details of head and flow rate required
e. Type and duration of the availability of the power supply
f. Selecting the operating speed of the pump and suitable drive/driving gear
g. The efficiency of the pump/s and consequent influence on power consumption and the running costs
h. Various options are possible by permuting the parameters of the pumping system, including the capacity and number of pumps including stand byes, combining them in series or parallel

5.7.1 Application of Specific Speed in Selection of Speed, Discharge, and Head

Specific speed is a very useful parameter in pump design, selection, determination of efficiency, the shape of H-Q, P-Q, and efficiency-Q characteristics, number of pumps, head per stage, suitability of the pump for required head range, selection of rpm.

These parameters are combined together in the term specific speed of a pump. The pressure and discharge of a pump vary with pump speed. A pump of a given geometrical design is characterised by specific speed N_s. This is the hypothetical speed of a geometrically similar pump with an impeller diameter D such that it will discharge a unit volume of flow against a unit head at maximum efficiency. It is expressed by the following formula.

$N_s = \; N\, Q^{0.5}/h^{0.75}$

Where

N_s = Pump specific speed

N = rpm

Q = pump discharge m^3/s, (US gpm); irrespective of single suction and double suction pump

h = head per stage, m(ft)

The conversion factor is 1 (SI) = 51.645 USCU.

However, most aspects of the performance characteristics of the different types of pumps can be determined based on their specific speed. Some useful observations are summarised below.

a. Figure 5.10 states values of specific speed versus efficiency. It is also seen that efficiency is higher for higher Q for the same specific speed. It is seen from the figure that better efficiency can be obtained if Ns is between 39 to 68 (SI)/2000 to 3500 (USCS).

b. Variable parameters are Q per pump, head per stage, and N:

- Ns is directly proportional to the square root of Q, i.e., $Q^{0.5}$. The discharge Q per pump can be varied by changing the number of pumps. This indicates that the number of the pumps should be minimum for better Ns.
- Ns is inversely proportional to 0.75 power of h (head per stage). Lesser the head per stage, Ns is higher.
- Ns is directly proportional to N (rpm). Thus, higher N renders better Ns.

- The objective should be to aim for Ns in the range 39 to 68 (SI)/2000-3500 USCS by varying the parameters Q, h, and N.
- In a single stage, high head pumps even if Ns is less than 39 (SI)/2000 USCS, there is no choice and lower Ns has to be accepted. However, if Ns is less than 42 (SI)/2170 USCS, the H-Q characteristic is unstable and not suitable for parallel operation.
- Ns above 80 (SI)/4100 USCS should be avoided as shut-off power is higher than the power required at the BEP, necessitating a higher motor rating for the pump. Such a pump cannot be started with the delivery valve closed which is an essential requisite for parallel operation.
- If Ns is less than 39 (SI) 2000 USCS, a head range beyond +7.5% is not probable and should be avoided to prevent heated operation at head higher than duty point/BEP.
- If Ns is less than 58 (SI)/3000 USCU, P-Q characteristics rise at higher Q. Motor for such pump need to be selected considering maximum power required corresponding to lowest head within specified head range.

c. Figure 5.10 illustrates the relationships between the pump efficiency, the shape of the impeller, and the nature of the curves of head (H) versus discharge (Q), power versus Q, and efficiency versus Q as influenced by the specific speed of the pump. The figure also helps in obtaining estimates of pump efficiency, which are useful in planning a pumping plant. This is applicable for all pumps including VT and submersible pumps.

d. Centrifugal pumps are generally with specific speeds above 36.

e. For high discharges, by which specific speed becomes high, the corresponding net positive suction head required also becomes high, the discharge is then shared by two impellers or two sides of an impeller as in a double suction pump.

   - For specific speed (Ns), full Q is to be considered even in respect of double suction pump. (This is based on the consideration that discharge collector is single and common for impeller having two/double entries on the suction side).
   - For suction specific speed (Nss) however, half Q is to be considered for a double suction pump. (This is based on the consideration that suction lift capability depends only on hydraulic losses on the suction side of the impeller).

f. Similarly, for high heads by which the specific speed becomes low, and hence the attainable efficiency becomes low, it can be arranged that the head is distributed amongst several impellers as in multistage pumps, thus improving the specific speed of each stage and consequently the attainable efficiency.

   - The NPSHr characteristic of a pump is parabolic, increasing with flow rate. Pumps of high specific speed have high NPSHr.

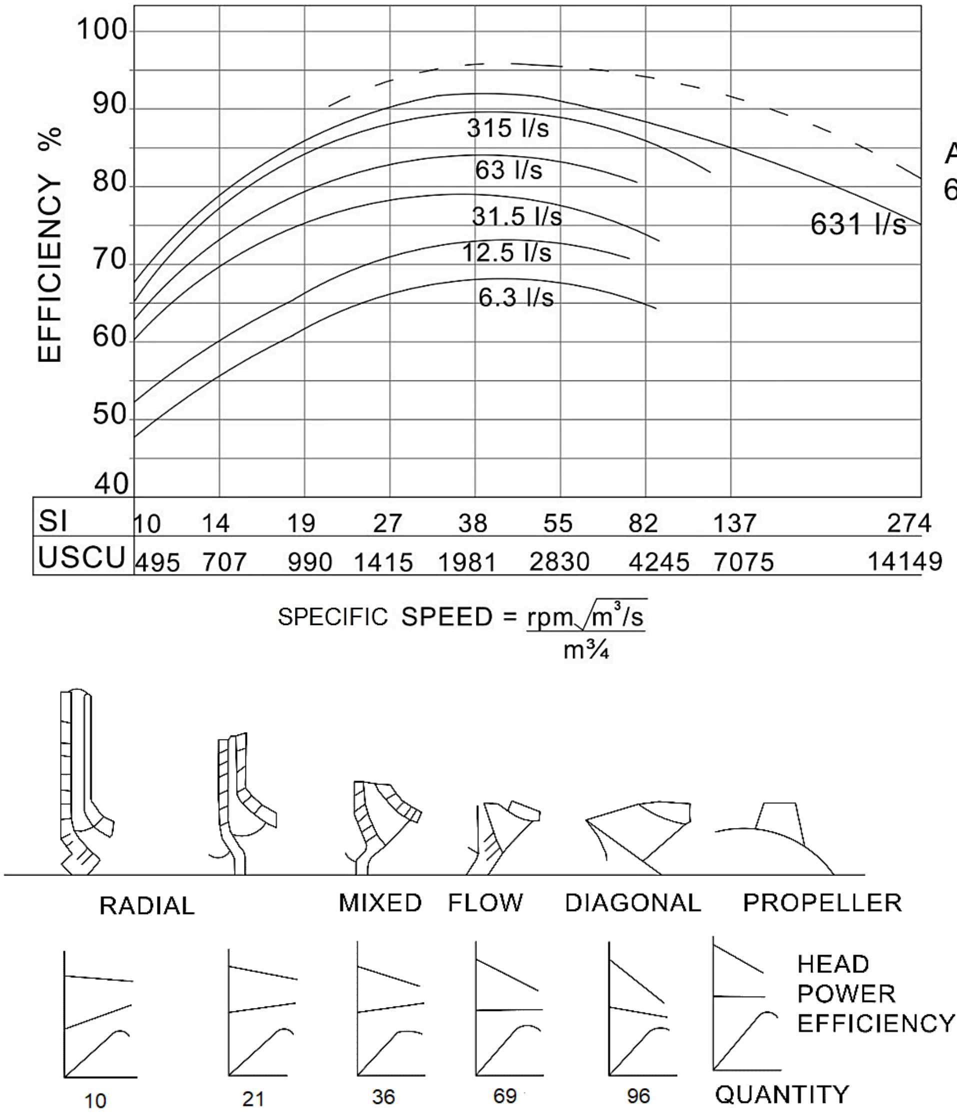


Figure 5.10: Specific Speed and Efficiency Characteristics

5.7.2 Considerations of the System Head Curve in Pump Selection

A pump or a set of pumps has to satisfy the needs of the pumping system. Hence one has to first evaluate the head needed to be developed by the pump for delivering different values of flow rate. A plot of these values is called the system head curve. Each point on the system head curve denotes the head comprised of the following:

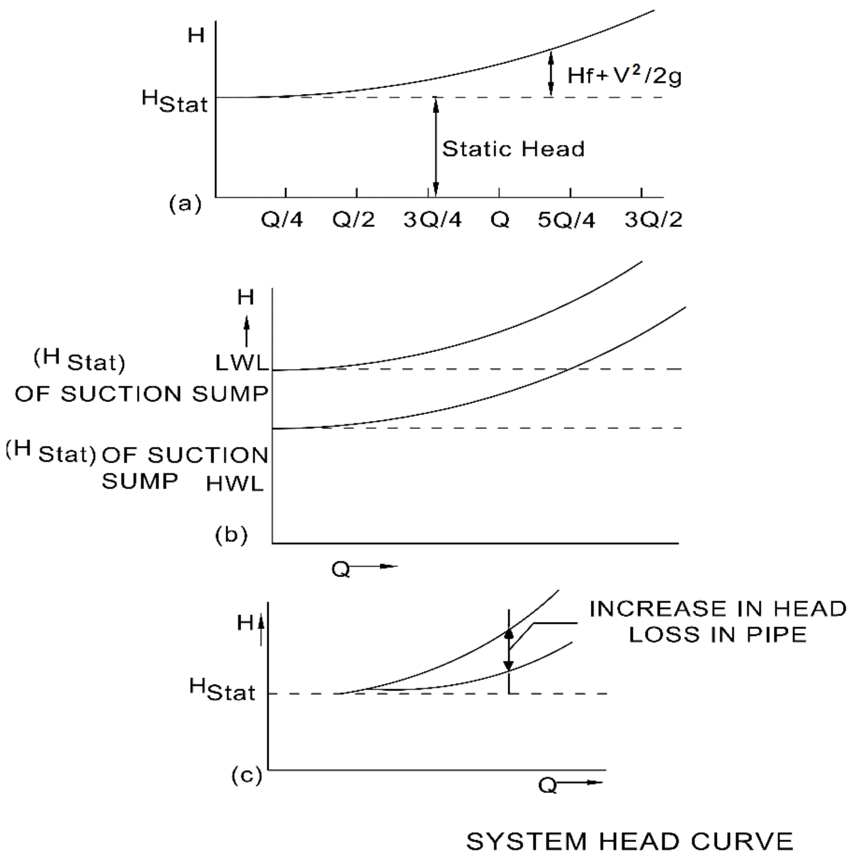


Figure 5.11: System Head Curve

The system head curve will change by any changes made in the system, such as a change in the length or size of the piping, change in size and/or the number of pipe fittings, changes in the size, number, and type of valves by operating the valves semi-open or fully open. These changes can cause the system head curve to be steep or flat as shown in Fig. 5.11 (c).

(i) Static suction lift/suction head

Static suction lift/suction head is the elevation difference from the centre line of the pump to the water level in the suction sump.

(ii) Static Delivery Head

Pump discharge is admitted at TWL/HWL in a tank by terminating the inlet pipe suitably and not at bottom of the tank. The static delivery head is the elevation difference from the centre line of the pump to the top of the exit pipe or TWL/HWL whichever is higher.

(iii) Static Head

This is the difference between the level of the liquid in the suction sump and the level of the highest point on the delivery piping. The static head is more at the low water level (LWL) and less at the high-water level (HWL). It is the sum of static suction lift/suction head and static delivery head.

(iv) Friction Head

This is the sum of the head losses in the entire length of the piping, from the foot valve to the final point of delivery piping, also the losses in all the valves, i.e., the foot valve, the non-return (reflux) valve, scour/wash out valves, air valve and the isolating (generally, sluice or butterfly) valves, and the losses in all pipe fittings such as the bends, tees, elbows, reducers, etc. Friction head also includes exit losses. Minor losses are generally about 10% of straight pipeline losses calculated as per Darcy-Weisbach or Hazen-Williams equations. The friction head varies particularly with the rate of flow. Details for calculating the friction heads are given in Chapter 6: Transmission of Water in Part A of this Manual.

(v) Velocity Head

Velocity head in exit loss, is of very small magnitude about 0.05-0.15 m and considered as part of minor losses. At the final point of delivery, the kinetic energy is lost to the atmosphere. To recover part of this loss, a bell mouth/flared outlet is often provided at the final point of delivery. The kinetic energy at the final point of delivery has also to be a part of the velocity head.

(vi) Station Losses

An additional component of the head is station losses on account of losses in foot valve, suction piping, fittings, suction valve, delivery/discharge piping, fitting, NRV/DPCV, SV/BFV, etc., and header. The magnitude of station losses is between 1.0 to 2.0 m. However, it is difficult to show station losses in the system head curve as it is the sum of losses in piping and valves, etc., of only one of the pumps (maximum to be considered) and losses for combined discharge in the header.

(vii) Total pump head

It is the sum of all heads listed above, viz., static head, friction head, velocity head, and station losses.

(viii) Operating Point

It is a point where the system head curve and H-Q curve intersect. Refer to Figure 5.12 and Section 5.9.

5.7.3 Summary View of Application Parameters and Suitability of Pumps

Based on the considerations in section 5.6, a summary view is compiled of the application parameters and suitability of pumps of various types and presented in Table 5.3. However, these are general guidelines. Specific designs may either not satisfy the limits or certain designs may exceed the limits. The stipulation regarding VFD compliance is based on present manufacturing features.

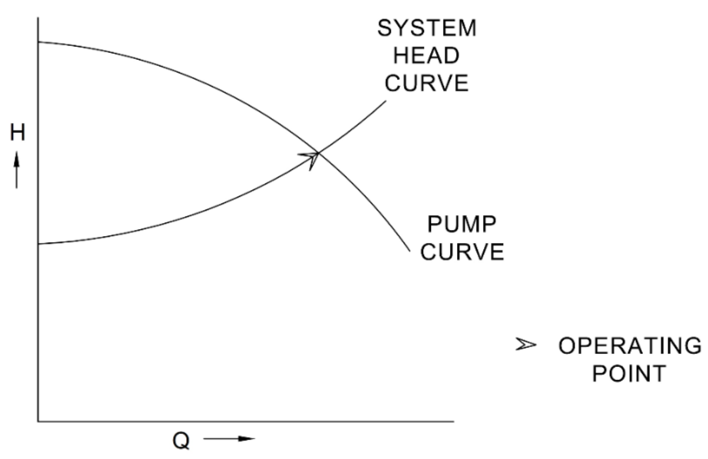


Figure 5.12: Operating Point of the Pump

Table 5.3: Application Parameters and Suitability of Pumps

Pump Type	**Suction-Capacity to lift**			**Head Range**			**Discharge Range**			**Application Features**							**Remarks**	
										Compatibility for Speed Control			Intake/Sump: Setting Depth of Pump Centreline from Pump mounting floor/Operation floor				Possibility of Operation despite Flooding pump-motor with electrical room above flood level/nearby safe place	Space Requirement (for Pumping Station)
										Fixed Speed	Variable Speed (VFD driven)							
	Low 3.5 m	Medium 6 m	High 8.5 m	Low Up to 10 m	Medium 10-40 m	High Above 40 m	Low Upton 30L/s (108 m^3/h)	Medium up to 500L/s (1,800 m^3/h)	High Above 500L/s (1,800m^3/h)		Motor portion	Pump portion	up to 3.5m deep	Above 3.5 to 7 m deep	Above 7 to 12m	beyond 12m deep		
Horizontal centrifugal end suction	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	No	Ok	needs VFD compatible motor	Ok	Ok	Ok subject to checking for safe operation	X	X	No	Large

Pump Type	Suction-Capacity to lift			Head Range			Discharge Range			Application Features							Remarks	
										Compatibility for Speed Control			Intake/Sump: Setting Depth of Pump Centreline from Pump mounting floor/Operation floor				Possibility of Operation despite Flooding pump-motor with electrical room above flood level/nearby safe place	Space Requirement (for Pumping Station)
										Fixed Speed	Variable Speed (VFD driven)							
	Low 3.5 m	Medium 6 m	High 8.5 m	Low Up to 10 m	Medium 10-40 m	High Above 40 m	Low Upton 30L/s (108 m^3/h)	Medium up to 500L/s (1,800 m^3/h)	High Above 500L/s (1,800m^3/h)		Motor portion	Pump portion	up to 3.5m deep	Above 3.5 to 7 m deep	Above 7 to 12m	beyond 12m deep		
Double suction Horizontal Split Casing	Ok	No	No	Ok	Ok	Ok	No	Ok	Ok	Ok	needs VFD compatible motor	Ok	Ok	Ok subject to checking for safe operation	X	X	No	Large
Horizontal Multistage Centrifugal	Ok	Ok	No	No	Ok	Ok	Ok	Ok	No	Ok	needs VFD compati	Ok	Ok	Ok subject to checki	X	X	No	Large

Pump Type	Suction-Capacity to lift			Head Range			Discharge Range			Application Features							Remarks	
										Compatibility for Speed Control			Intake/Sump: Setting Depth of Pump Centreline from Pump mounting floor/Operation floor				Possibility of Operation despite Flooding pump-motor with electrical room above flood level/nearby safe place	Space Requirement (for Pumping Station)
											Variable Speed (VFD driven)							
	Low 3.5 m	Medium 6 m	High 8.5 m	Low Up to 10 m	Medium 10-40 m	High Above 40 m	Low Upton 30L/s (108 m^3/h)	Medium up to 500L/s (1,800 m^3/h)	High Above 500L/s (1,800m^3/h)	Fixed Speed	Motor portion	Pump portion	up to 3.5m deep	Above 3.5 to 7 m deep	Above 7 to 12m	beyond 12m deep		
											ble motor			ng for safe operation				
Submerged Centrifugal	When suction lift is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Medium
Jet Pump	When Limitations of suction lift are to be overcome			Ok	Ok	No	Ok	No	No	Ok	x	Ok	Ok	Ok	Ok	Ok	Risky	Small

Pump Type	Suction-Capacity to lift			Head Range			Discharge Range			Application Features							Remarks	
										Compatibility for Speed Control			Intake/Sump: Setting Depth of Pump Centreline from Pump mounting floor/Operation floor				Possibility of Operation despite Flooding pump-motor with electrical room above flood level/nearby safe place	Space Requirement (for Pumping Station)
											Variable Speed (VFD driven)							
	Low 3.5 m	Medium 6 m	High 8.5 m	Low Up to 10 m	Medium 10-40 m	High Above 40 m	Low Upton 30L/s (108 m^3/h)	Medium up to 500L/s (1,800 m^3/h)	High Above 500L/s (1,800m^3/h)	Fixed Speed	Motor portion	Pump portion	up to 3.5m deep	Above 3.5 to 7 m deep	Above 7 to 12m	beyond 12m deep		
Vertical Turbine (Conventional)	When suction lift is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok	Ok	needs VFD compatible motor	Ok subject to choking line shaft diameter and bearing spacing for	Ok	Ok	Ok	Ok subject to checking line shaft diameter and bearing spacing for	Risky	Medium

Pump Type	Suction-Capacity to lift			Head Range			Discharge Range			Application Features							Remarks	
										Compatibility for Speed Control			Intake/Sump: Setting Depth of Pump Centreline from Pump mounting floor/Operation floor					
											Variable Speed (VFD driven)							
	Low 3.5 m	Medium 6 m	High 8.5 m	Low Up to 10 m	Medium 10-40 m	High Above 40 m	Low Upton 30L/s (108 m^3/h)	Medium up to 500L/s (1,800 m^3/h)	High Above 500L/s (1,800m^3/h)	Fixed Speed	Motor portion	Pump portion	up to 3.5m deep	Above 3.5 to 7 m deep	Above 7 to 12m	beyond 12m deep	Possibility of Operation despite Flooding pump-motor with electrical room above flood level/nearby safe place	Space Requirement (for Pumping Station)
												safe critical speed				safe critical speed		
Submerged Turbine	When suction lift is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Ok	Compact
Submersible pump(conventional)-or Polder	When suction lift is to be avoided			Ok	Ok	Ok	Ok	Ok	Ok	Ok	needs VFD compati	Ok	Ok	Ok	Ok	Ok	Ok	Very small

Pump Type	Suction-Capacity to lift			Head Range			Discharge Range			Application Features							Remarks	
										Compatibility for Speed Control			Intake/Sump Setting Depth of Pump Centreline from Pump mounting floor/Operation floor				Possibility of Operation despite Flooding pump-motor with electrical room above flood level/nearby safe place	Space Requirement (for Pumping Station)
										Fixed Speed	Variable Speed (VFD driven)							
	Low 3.5 m	Medium 6 m	High 8.5 m	Low Up to 10 m	Medium 10-40 m	High Above 40 m	Low Upton 30L/s (108 m^3/h)	Medium up to 500L/s (1,800 m^3/h)	High Above 500L/s (1,800m^3/h)		Motor portion	Pump portion	up to 3.5m deep	Above 3.5 to 7 m deep	Above 7 to 12m	beyond 12m deep		
											ble motor							
Positive displacement pumps	Normally self-priming			Limited only by the pressure which casing can withstand			Ok	Ok	Ok	Ok	needs VFD compatible motor	Ok	Ok	X	X	X	No	Medium
							Easy adaptation for dosing or metering			Ok								

5.7.4 Consideration while Selecting Pump for Series or Parallel Operation

(i) When pumps are to run in parallel, to obtain the combined H-Q characteristics, for different values of the head, the values of the flow of individual pumps are to be found and to be added (See Fig. 5.13). The system head curve then intersects the combined H-Q characteristics at higher head and discharge. Each pump ought to be capable of developing such a high head, that too within its zone of stability. Rather, it is always desirable to put into parallel operation only pumps having stable H-Q characteristics.

(ii) A pumping system is often sought to be modified to meet the increasing demand by commissioning additional pumps in parallel. It must be noted however that because the system head curve intersects the combined H-Q curve at a point having the head also higher, an additional pump would not increase the discharge proportionately, i.e., by making two identical pumps work in parallel when one is previously operative, the discharge would not double. (Fig. 5.14)

(iii) Conversely, if a system is to run with many pumps in parallel but is modified to run with only a few of the pumps as in summer, for example, then the duty flow of each pump becomes more than when all the pumps are running. The individual pump would demand higher NPSHr at the higher duty flow. If the NPSHa would not be adequate, the pump/s would cavitate. To prevent such a possibility, individual pumps, which are to be put into parallel operation, would be so selected that the duty flow of combined parallel operation would be to the left of the BEP of the individual pump. By this, when only a few pumps are to run, the duty flow of the individual pump would shift to the higher flow nearer to its BEP (Fig. 5.15)

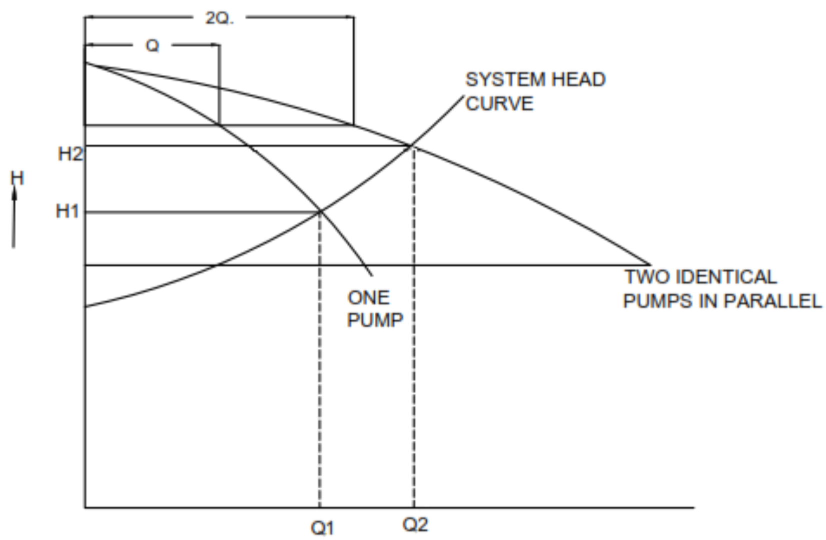
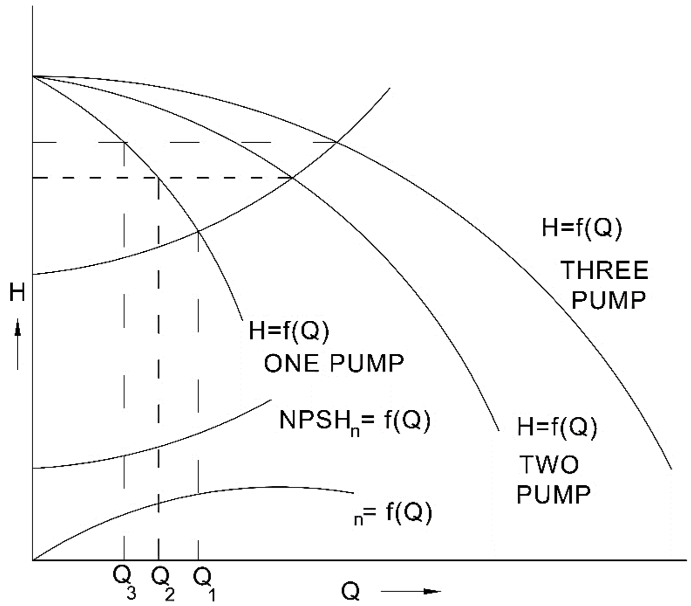


Figure 5.13: Combined Characteristics of Two Pumps in Parallel



NO. OF PUMPS IN OPERN.	FLOW OF EACH PUMP.
1	Q_1
2	Q_2
3	Q_3

Figure 5.14: One Or More Pumps in Parallel

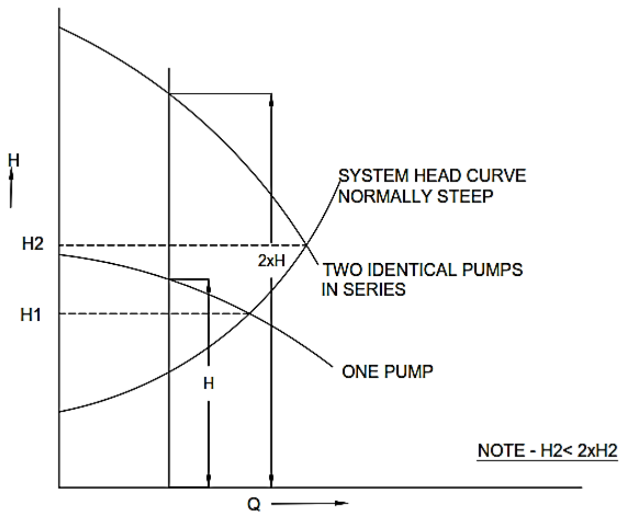


Figure 5.15: Series Operation of Pumps

(iv) Pumps in series are similar to multistage pumps. Rather, multistage pumps are only a compact construction, where series operation is in-built. To obtain the combined H-Q characteristics of pumps in series, for various values of discharge, the values of the head from the H-Q characteristics of individual pumps are to be noted and added. The system head curve would intersect the combined H-Q curve at a point of higher head and discharge (See Fig. 5.15). The individual pumping, in this case, ought to be capable of giving the higher discharge.

(v) If the system head curve comprises a high static head and a flat curve, the intersection at higher discharge on the combined H-Q characteristics may be at such discharge where the NPSHr of the individual pump would be high and the pump/s may cavitate.

(vi) Series operation is most appropriate, where the system head curve is steep. For the pumps to be put in series operation, each pump should be capable of withstanding the highest pressure that is likely to be developed in the system. (Fig. 5.15)

The head towards the potential difference between the centreline of one pump and the suction of the next pump, plus the friction losses in the pipeline between the deliveries of one pump up to the suction of the next pump has to be considered as a part of the total head of the pump giving the delivery. In a series system, the total head of each pump may have to be individually calculated, especially when the features contributing to head calculations are significantly different, as in the case of booster stations along a long conveyance pipeline.

5.7.5 Considerations of the Size of the System and the Number of Pumps

For small pumping systems, generally of capacity less than 25 MLD, two pumps of average daily discharge (one duty and one standby) should be provided. Alternatively, two duty and one standby, each of 50% of average daily water demand may be provided. Although this alternative would need larger space, it facilitates flexibility in regulating the water supply. Also, in an emergency of two pumps going out of order simultaneously, the third helps to maintain at least partial supply.

The strategy for the number of working and standby pumps is proposed as follows.

i. The number of working pumps shall be decided such that the specific speed (Ns) of pumps is within optimum efficiency range inferred after detailed analysis using the following approach:
   The objective shall be to arrive at optimum Ns by varying parameters Q per pump, speed, and head per stage of the pump as follows:
   a) by varying number of pumps, thus, varying Q per pump
   b) by varying rpm to standard values
   c) by varying number of stages, thus, varying head per stage
   d) Ns is directly proportional to rpm
   e) Ns is proportional to the square root of Q per pump
   f) Ns is inversely proportional to the 0.75 power of the head
   g) rpm is generally, 980, 1480; in some cases, 590, 740, 2900
   h) If rpm increases, wear and tear increase, but pump size and, therefore, pump cost, reduces and vice versa
   i) If the number of stages is reduced, head per stage increases and Ns reduces and vice versa
   j) The number of stages for VT pump should generally not exceed five; in exceptional cases, up to 10
ii. There is no ideal solution. At the most, the best solution is perhaps possible. The solution to be concluded is usually a compromise between conflicting requirements. Even the chosen solution is not free from demerits.
iii. Based on the above, the number of working pumps is decided.
iv. If the number of working pumps is one, then, a combination
   - 1(W) + 1 (S).
   If number of working pumps are two, then,
   - Generally, 2 (W) + 1 (S)
   - Large scheme 2(W) + 2 (S)

This is based on the consideration that one pump may be under major repairs and the 2nd pump is under minor repairs and out of service for a few hours or one or two days. Also, the water supply service level cannot be reduced.

If the number of working pumps is three to five, then, a minimum of two numbers on standby, i.e.,

- 3(W) +2(S);
- 4(W) + 2(S);
- 5(W) + 2(S).

If the number of working pumps is 6 to 10, then, a minimum of three standby pumps shall be provided.

5.7.6 Considerations Regarding Probable Variations of Actual Duties

5.7.6.1 Affinity Laws

The running speed of the electric induction motors is at a slip from its synchronous speed. The running speed of the motor is also influenced by variations in the supply frequency. Since the pump characteristics furnished by the pump manufacturers are at a certain nominal speed, depending upon the actual speed while running, the actual pump performance would be different from the declared characteristics. Estimates of the pump performance in actual running can be worked out from the declared characteristics, by using the following affinity laws.

If

$$\frac{n''}{n'} = k, then\ \frac{Q''}{Q'} = k;$$

$$\frac{H''}{H'} = k^2, and\ \frac{P''}{P'} = k^3;$$

In the above formulas, n denotes the speed of the pump, P denotes the power input to the pump, the superscript “denotes the values at the actual speed and the superscript ' denotes the values at the nominal speed.

Recalculating the pump performance at the actual speed would reveal the following.

(a) If the actual speed is less than the nominal speed, then the values of the discharge, head, and power required to be input to the pump would all be less than the values from the declared characteristics.

(b) Similarly, if the actual speed is more than the nominal, it should be checked that the higher power input required would not overload the motor.

(c) When the actual speed is more because the discharge is also correspondingly more, the NPSHr would also be more, varying approximately as per the following formula.

$$\frac{NPSHr''}{NPSHr'} = k^2$$

5.7.6.2 Scope for Adjusting the Actual Characteristics

To avoid overloading or cavitation, marginal adjustment to the pump performance may be done at the site, either by employing speed-change arrangements or by trimming down the impeller. The modifications in the performance on trimming the impeller can be estimated using the following relations:

If, $\frac{D''}{D'} = k, then\ \frac{Q''}{Q'} = k; \frac{H''}{H'} = k^2, then\ \frac{P''}{P'} = k^3;$

Such modifications are recommended to be done within 5 to 20 per cent of the largest diameter of the impeller. The percentage depends upon the design, size, and shape of the impeller. Generally, a reduction in diameter is allowable within 10 to 20 per cent of the maximum impeller diameter of the pump in radial flow impellers and 5 to 15 per cent in mixed flow and axial flow impellers. The pump manufacturer should be consulted on this reduction.

5.8 Consideration of the Suction Lift Capacity in Pump Selection

5.8.1 Significance of NPSHr

The suction lift capacity of a pump depends upon its NPSHr characteristics. Significance of NPSHr can be explained by considering an installation of a pump working under a suction lift as illustrated in Fig. 5.16.

When a pump, installed as shown is primed and started, it throws away the priming water and has a vacuum developed at its suction. The atmospheric pressure acting on the water in the suction sump then pushes the water through the foot valve, into the suction line, raising it up to the suction of the pump. While reaching up to the suction of the pump, the energy content of the water, which was one atmosphere when it was pushed through the foot valve, would have reduced, partly in overcoming the friction through the foot valve and the piping and the pipe fittings, partly in achieving the kinetic energy appropriate to the velocity in the suction pipe, and partly in rising up the static suction lift. The energy content left over in the water at the suction face of the pump is thus less than one atmosphere until here the flow is fairly streamlined. But with the impeller rotating at the pump suction, the flow suffers turbulences and shocks and will have to lose more energy in the process. This tax on the energy of the water demanded by the pump, before the pump would impart its energy, is called the NPSHr of the pump.

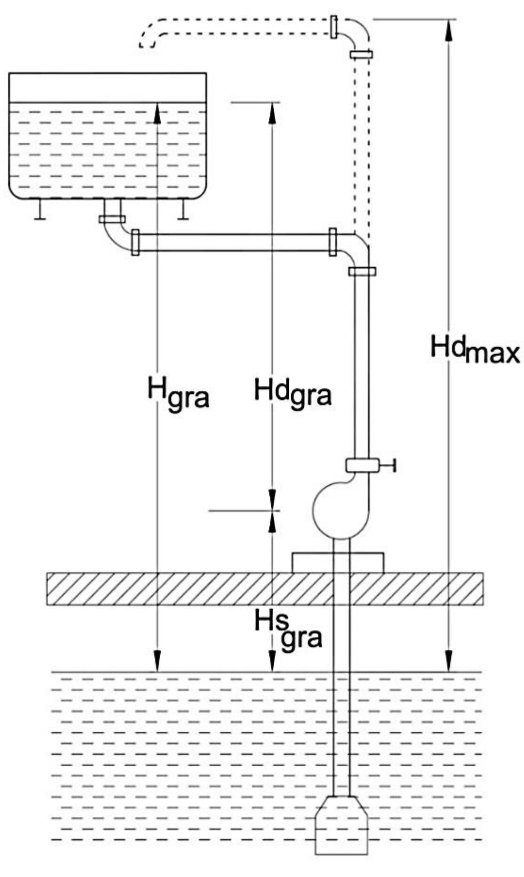


Figure 5.16: Illustration of Suction Lift and Static Delivery Head

5.8.2 Vapour Pressure and Cavitation

The energy of the water at the pump suction, even after deducting the NPSHr, should be more than the vapour pressure Vp corresponding to the pumping temperature. The vapour pressures in metres of water column (mWC) for water at different temperatures in degrees Celsius are given in Table 5.4.

Table 5.4: Vapour Pressure of Water

°C	mWC
0	0.054
5	0.092
10	0.125
15	0.177
20	0.238
25	0.329
30	0.427
35	0.579
40	0.762
45	1.006
50	1.281

If the energy of the water at the pump suction would be less than the vapour pressure, then the water would tend to evaporate. Vapour bubbles so formed will travel entrained in the flow until they collapse. This phenomenon is known as cavitation. In a badly designed pumping systems, cavitation can cause extensive damage to suction side of impeller and suction casing due to erosion, pitting and the vibration and noise associated with the collapsing of the vapour bubbles.

5.8.3 Calculating NPSHa

To insure against cavitation, the pumping system has to be so devised that the water at the pump suction will have adequate energy. Providing for this is called as providing adequate net positive suction head available (NPSHa). The formula for NPSHa, hence becomes as follows.

NPSHa = $P_S - Hf_s - \frac{V_s^2}{2g} - Z_S - V_p$

Where,

P_S = Suction Pressure in the absolute unit.

If the suction tank is open to the atmosphere, Ps = Atmospheric pressure in mWC corresponding to altitude

Hf_s = Frictional Losses across the Foot Valve, Suction Piping, and Fittings

V_s = Velocity head at the suction face

Z_S = The potential energy corresponding to the difference between the levels of the impeller eye centreline and of the water in the suction sump

V_p = The vapour pressure

While calculating NPSHa, the atmospheric pressure at the site should be considered, as the atmospheric pressure is influenced by the altitude of the place from the mean sea level (MSL). Data on the atmospheric pressure in mWC for different altitudes from MSL is given in Table 5.5.

Table 5.5: Atmospheric Pressure in mWC at Different Altitudes above MSL

Altitude above MSL in m	mWC
up to 500	10.3
1,000	9.8
1,500	9.3
2,000	8.8
2,500	8.3
3,000	7.8
3,500	7.3
4,000	6.8

NPSHa is not characteristic of a Pump; it depends on pump installation level with respect to WL in sump/intake, atmospheric pressure, losses in suction piping, and vapour pressure of water at water temperature. NPSHa is an important parameter to calculate suction specific speed which indicates suitability for cavitation-free operation for site suction conditions or otherwise as elaborated in discussions on suction specific speed.

5.8.4 Suction Specific Speed and its application for suitability for Suction head

The formula for suction specific speed (Nsss), is given by,

$$Nsss = \frac{N\sqrt{Q}}{NPSHa^{0.75}}$$

Where

N = rpm

Q = discharge per suction side of impeller (for double suction impeller half Q to be considered); m^3/s (SI)/US gpm (USCU)

NPSHa = Net positive suction head available; m (SI)/feet (USCU)

The method for calculating NPSHa is detailed in subsection 5.8.3 as per which NPSHa depends on site installation, atmospheric pressure at site altitude, and vapour pressure at maximum water temperature at site temperature. Thus, NPSHa for the same pump shall be different for installations at two different sites.

Application of suction specific speed:

Suction specific speed (Nsss) is a very useful parameter for concluding the suitability of pump for prevailing suction conditions at site environments and installation level as elaborated in (a) above.

Nsss should not exceed 145 (SI)/7,500 USCU for pump installation to achieve cavitation-free operation under the site conditions of atmospheric pressure and maximum water temperature.

5.8.5 Guidelines On NPSHr

The NPSHa has to be so provided in the systems that it would be higher than the NPSHr of the pump. The characteristics of the pump's NPSHr are to be obtained from the pump manufacturers. However, some general guidelines for maximum suction lift or min. NPSHa based on the type of a pump and based on the range of head and the specific speed is compiled below:

General Observations

a. In some cases, horizontal centrifugal pumps are installed with a suction lift.

b. For vertical pumps, mainly of the vertical turbine type, and of the borewell submersible type, the suction lift has to be totally avoided. Even for these pumps, when the discharge required is high, they have to be installed providing the minimum submergence. The minimum submergence required may at times demand submerging more than the first stage of the pump. It should also be checked whether the submergence would be adequate for vortex-free operation.

c. Jet centrifugal combinations can work for lifting from depths up to 70 m. However, the efficiency of the pumps is very low.

d. Positive displacement pumps are normally self-priming. However, this should not be confused with the NPSHr. Even if the NPSHa is not adequate, the pump may prime itself and run, but would cavitate.

5.9 Defining the Operating Point or the Operating Range of a Pump

The operating point of a pump is the point of intersection of the system head curve with the H versus Q characteristics of the pump. Shifting of the system. Head curve will cause a change in the operating point of the pump. Hence, the following points are worth noting.

a) If the level of water in the suction sump would deplete during pumping from HWL to LWL the operating point of the pump would vary from a low-head-high discharge point to a high head low-discharge point (Fig. 5.17).

b) If in a pumping system, the throttling of the delivery valve from fully open to close, shifts the system head curve from a flat curve, intersecting the pumps H-Q curve at high flow initially to a steep system head curve intersecting the pumps H-Q curve at the high head (Fig. 5.18).

   Similarly, a pumping system can be with a flat or steep system head curve depending on relative magnitudes of static head and friction losses in pumping mains.

   The most average water level in the suction sump and the most average system head curve for designed number of duty pumps would define the operating point of the pump. For such an operating point of the pump, the pump should have its point of maximum efficiency at or nearest to it. To provide for marginal changes in the operating point, e.g., between HWL and LWL, the nature of the efficiency characteristics of the pump should be as flat as possible in the vicinity of the point of its best efficiency, often called as the BEP.

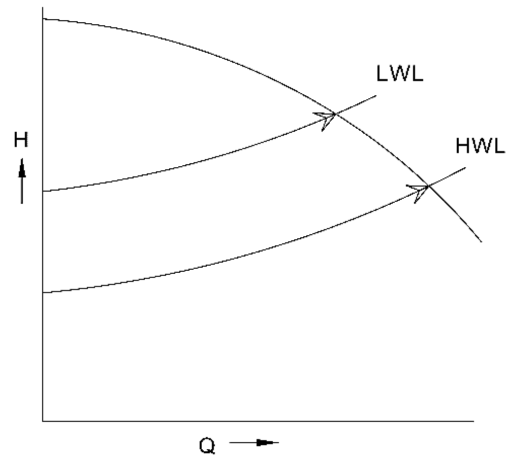


Figure 5.17: Change in Operating Point of Pump with Change in Water Level in Suction Pump

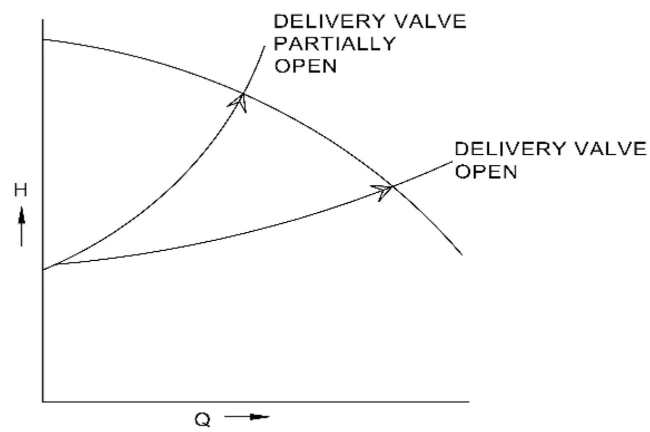


Figure 5.18: Change in Operating Point of Pump due to Throttling of Delivery Valve

c) When specifying the operating point of the pump, margins, and safety factors, especially in specifying the head should be avoided. On providing margins and safety factors, the rated head for the pump would work out high. In actual running, the pump would work at a head less than the rated head and yield high discharge. It would be noted that the power versus Q characteristics of pumps of specific speeds up to 29 (SI)/1500 USCS is with positive gradient, hence, demanding more power at higher discharge. With such higher power demand, the drive may get overloaded.

d) By working at high discharge, the NPSHr demanded by the pump would be higher. If NPSHa is not adequate for this higher NPSHr, the pump may cavitate.

Due to the high discharge included, the pump may vibrate. Sometimes this may result in serious damage to the shaft and bearings.

e) Operating/duty point and operating head range pump parameters are to be specified as under:

i. Duty Point: Q, H, minimum acceptable efficiency, and maximum suction lift as per pump installation levels and LWL.
ii. Head range:
   a. VT pumps, submerged VT and submersible pumps:

- +10% and -25% of duty head;
- actual variation in the head from a solo operation to parallel operation up to the maximum number of working pumps and WL variation from LWL to TWL; or
- ±3 m whichever is the highest amongst three bullet values for the maximum head and the lowest for the minimum head.

b. Centrifugal pumps and submerged centrifugal:
   - generally, +10% and -25% of duty head; if shut-off head is within +15%, then +7.5% and -25% of duty head;
   - actual variation in the head from a solo operation to parallel operation up to the maximum number of working pumps and WL variation from LWL to TWL; or
   - ± 3 m whichever is the highest amongst three bullet values for the maximum head and the lowest for the minimum head.

5.10 Stability Of Pump Characteristics

In the H-Q characteristic of the centrifugal pump, the flow reduces as the head increases. If the head increases continuously until zero flow or until full close, i.e., shut-off of the delivery valve, the H-Q characteristic is said to be stable. However, it is also probable that the shut-off head of a pump may be less than the maximum head, as shown in Figure 5.19 which may be realised at some positive flow. Such a characteristic of a pump is called an unstable characteristic. When operating such a pump at any head between the shut-off head and the maximum head, the flow will keep hunting between two values. Because of this, the performance of the pump becomes erratic and unstable.

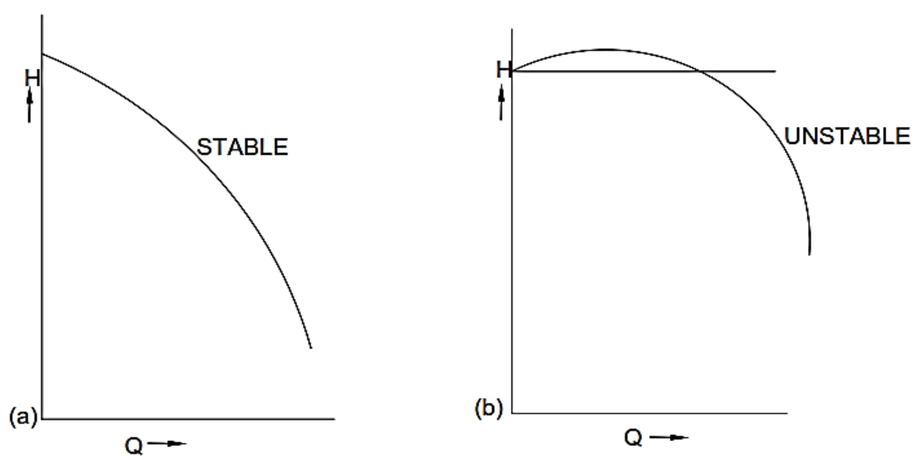


Figure 5.19: Stable and Unstable Characteristics of Pump

While selecting a pump, it ought to be checked that the highest head by the intersection of the system head curve would be less than the shut-off head, in the case of a pump with unstable characteristics. For multiple pumps in parallel, pumps with stable H-Q characteristics should only be selected. If pumps have an unstable characteristic, one pump may operate at a rated discharge, but other pumps shall operate at a lower discharge point or may hunt between two discharge points.

5.11 Important Guidelines for Pump Selection

i. The variable parameters N (rpm), h (head per stage) by varying number of stages, and Q by varying number of working pumps, should be such that the specific speed of pumps is within the range of 38 to 68 (SI) or 2,000 to 3,500 USCU for optimum efficiency. Under no case specific speed should exceed 80 (SI)/4,100 USCS as shut-off power is higher than BEP power, requiring a higher kW motor for starting the pump.
ii. N should preferably be 980 rpm for large installation. In exceptional cases, 1,480 rpm so as to restrict wear and tear to a minimum and limit noise and vibration levels. 2,900 rpm should be avoided to the possible extent except for borewell type submersible pumps.
iii. In the case of VT or multistage centrifugal pumps, the number of stages should not generally exceed five. However, in exceptional cases, stages up to a maximum of 10 numbers can be accepted.
iv. The centrifugal pump should preferably be a double suction horizontal split casing. End-suction pump should generally be avoided.
v. Submersible pumps for the open well, intake, and sump should generally be avoided as their lifespan is much less than 5-10 years.
vi. In some cases, accept lower rpm, i.e., 740 or 590 or 490 though the cost is much higher.
vii. Use of oil lubricated VT pump should be restricted to river water turbidity higher than 500 NTU.
viii. The diameter of column pipes should be decided by the client as this parameter is not dependent on bowl assembly design and shall be on basis of velocity 1.75 to 2.75 m/s; lower value for low Q and higher value for medium/high value of Q.
ix. The diameter of the bell mouth shall be on basis of 1.2 to 1.4 m/s entrance velocity.
x. The critical speed of the pump/impeller shaft should not be within 75-125% of the rpm of the pump.
xi. Subsurface delivery VT pumps should be avoided as hydraulic thrust is encountered at delivery tee connection vulnerable to column assembly misalignment.
xii. Thrust bearing of conventional VT pump shall be suitable for 40,000-50,000 hours of life.

5.12 Motor Rating

After the operating point of a pump is decided as discussed in 5.9, the efficiency of the pump can be estimated. The rating of the drive should be such that it would not get overloaded when the pump would be delivering the high discharge, as with HWL in the suction sump. Also, the drive rating should be adequate to provide for the negative tolerance on efficiency and the positive tolerance on discharge, applicable for variations in actual pump performance from the rated performance.

The power needed as input to the pump is the power output by the drive, i.e., at the pump shaft. Since most drives are coupled directly to the pump, the power at the pump shaft denotes the brake power of the drive. All drives are rated only as per their brake power capacity, often quoted in brake kilowatt~~s~~ (bkW).

Input to pump and motor is given by the formulas given as under:

i. Pump bkW = $\frac{\text{g x Q x H x Spg}}{effp}$

Where

g = 9.81 m/s^2 (as acceleration due to gravity)

Q = discharge, m^3/s

H = pump head, m

Spg = specific gravity of liquid

effp = pump efficiency

bkW = brake kilowatt input to pump

Specific gravity for water is 1

ii. Input to motor = Pump bkW/effm
Where

effm = motor efficiency

To provide margins over the bkW required at the operating point and maximum bkW required over the required head range, so that the overloading would not happen, the following margins (Table 5.6) are recommended.

Table 5.6: Margins to Decide Motor Rating

bkW required at the operating point	Percentage Margin
Up to 1.5	50%
1.5 to 3.7	40%
3.7 to 7.5	30%
7.5 to 15	20%
15 to 75	15%
above 75	10%

5.13 Pump Testing

The objective of pump testing is to verify that the performance characteristics of the pump are appropriate for the service desired.

The testing is done both at the manufacturers' works and only for preventive maintenance in the field, with the following limitations:

As per IS 9137 for Class C test and IS 10981 for Class B test relating to testing of pumps, the standard arrangements and procedures described are those to be employed for testing a pump individually, without reference to its final installation conditions or the effect upon it of any associated fittings, these being the usual conditions in which a pump is tested at the manufacturer’s works. Acceptance tests can be carried out either at the manufacturer's workplace or at a place to be mutually agreed upon between the manufacturer and the purchaser.

5.13.1 Testing at Manufacturer’s Place

Since the testing at the manufacturers' place is done with water under ambient conditions, the duties desired with service-fluid have to be translated to equivalent duties with water under ambient conditions. Please refer to standards on testing, viz., IS: 9137 or IS: 10981 for permissible tolerances for the variation of test results from guaranteed duties. Out of these two standards, IS: 9137 details Class C code of testing, and IS 10981 details Class B code of testing. The Class B code of testing specifies a narrower band for tolerance, the implicit stringency affects both the cost and the period of delivery. The Class C code of testing is the most widely followed and adequate in most of the cases. However, for a pump above 225 kW, the Class B test is desirable.

The scheme of testing includes taking readings, doing calculations, and plotting:

- the H-Q characteristics;
- the P-Q characteristics; and

- the efficiency versus Q characteristics.
- Check for permissible unbalance for pumps above 150 kW which are discussed in the next sub section.

The actual speed of the shaft at the time of each reading would be different from the nominal speed. The value of the total head-flow rate and power input are to be converted to the nominal speed, using the affinity laws.

The readings of power input noted during testing are often the values of power input to the motor. Values of power input to the pump have to be derived by multiplying the values of power input to the motor with the appropriate values of motor efficiency.

For the values of motor efficiency, a reference has to be made to the motor characteristics. Often, these are available as motor output to the motor efficiency relationship. Since the readings during the test are for the motor input, the motor characteristics need to be converted into the appropriate motor input to the motor efficiency relationship.

After the performance characteristics are plotted, an assessment has to be made to check whether the plotting reveals variations from the guaranteed duties. The pump can be approved if the variations are within the permissible limits.

It may be noted that the limits specified in IS: 9137 and IS: 10981 give limits both for positive and negative variances.

Only occasionally the testing is extended to cover testing the NPSHr characteristics of the pump. Care is always to be taken to provide NPSHa such that it has an adequate margin over NPSHr at all flow rates in the operating range. Hence the data of NPSHr provided by the manufacturer need not be verified by an actual test. This is so advocated considering that:

- Conducting test for NPSHr requires elaborate and often special arrangements on the test bed and becomes costly and time-consuming.
- Even on readily available test rigs, the actual conducting of the test itself becomes time-consuming, exerting and with a cost element.
- The variations from the declared data are mostly on the safer side.

However, if the site plan is laden with such constraints that NPSHa cannot have adequate margins over NPSHr, then testing for NPSHr may be stipulated very clearly in the purchase specifications. Unless stipulated, routine testing of a pump does not include the test for NPSHr in the scope,

5.13.2 Balancing test for Impeller or rotating assembly

The pump impeller balancing is performed based on ISO 1940-1 for pumps above 150 kW. During carrying out the test at the manufacturer's place, the inspector shall verify the approved balancing test procedure and identify the following information:

- Speed (RPM)
- Acceptance Criteria (permissible unbalance)

Permissible residual unbalance

The permissible residual unbalance U_{per} can be derived based on a selected balance quality grade G by the following equation:

$$U_{per} = 1000\frac{(\, e_{per}\; X\; \Omega\,)\; X\; m}{\Omega}$$

Where

U_{per} is the numerical value of the permissible residual unbalance, expressed in gram millimetres (g·mm);

$(e_{per} X \Omega)$ is the numerical value of the selected balance quality grade, expressed in millimetres per second (mm/s); this is as per Table 1 of ISO 1940-1 (balance quality grades) 6.3 mm/s for pumps

m is the numerical value of the rotor mass, expressed in kilograms (kg);

Ω is the numerical value of the angular velocity of the service speed, expressed in radians per second (rad/s), with $\Omega = \frac{\pi \times n}{30} \approx \frac{n}{10}$ and the service speed n in revolutions per minute (r/min).

For example, if you have a 2 kg impeller with a 3,000-rpm rotor, the permissible unbalance is as follows:

$$\text{Permissible residual Unbalance} = U_{per} = 1000\frac{(2.5)\,X\,2}{314.2} = 15.91\ \text{g.mm}$$

The 2.5 is the ISO 1940-1 “grade of balance”. $(e_{per} X \Omega)$

Ω = Divide 3,000 RPM to 30/π to obtain speed in rad/s = 314.2

5.13.3 Testing at Site

At the site, the testing is done soon after installation to assess whether any adjustments are required to the pump characteristics. Further testing is done at the site, mostly once in a year to assess whether there is any deterioration in the performance of the pump due to wear and tear.

The objective of the field test is to serve as a timely caution for preventive maintenance and not one of obtaining very elaborate details of the pump characteristics.

During the testing at the site, it is often impractical to provide adequate instrumentation of an appropriate class of accuracy. Setting up the instrumentation may disrupt the online operation of the pump. Apart from the disruption, certain temporary modifications may be needed to introduce flow-measuring devices like the orifice plates, etc., in the line. A field test has to be scheduled considering when the disruption of the online operation can be tolerated.

5.14 Installation of Pumps

The procedure of installation depends upon whether the pump is to be mounted horizontal or vertical. Most pumps to be mounted horizontally are supplied by the manufacturers as a wholesome, fully assembled unit. However, pumps to be mounted vertically are supplied as sub-assemblies. For the installation of these pumps, the proper sequence of assembly has to be clearly understood from the manufacturer's drawings.

The installation of a pump should proceed through five stages in the following order:

i. Preparing the foundation and locating the foundation bolts.
ii. Locating the pump on the foundation bolts, however, resting on levelling wedges, which permits not only easy levelling but also space for filling in the grout later on.
iii. Levelling the pump.
iv. Applying grouting.
v. Performing alignment.

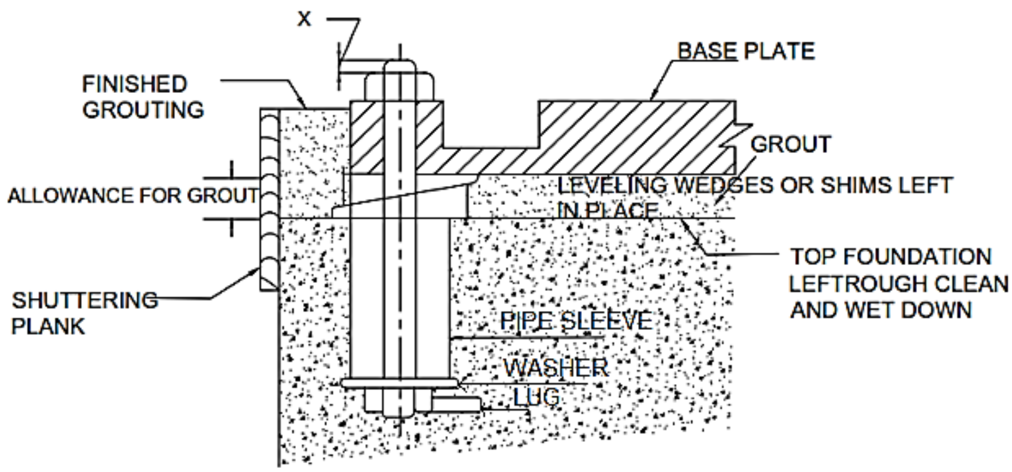


Figure 5.20: Typical Foundation Design

The following points should be taken care while installation:

(a) The foundation should be sufficiently substantial to absorb vibrations and form a permanent rigid support for the base plate. A typical foundation is illustrated in Fig. 5.20.

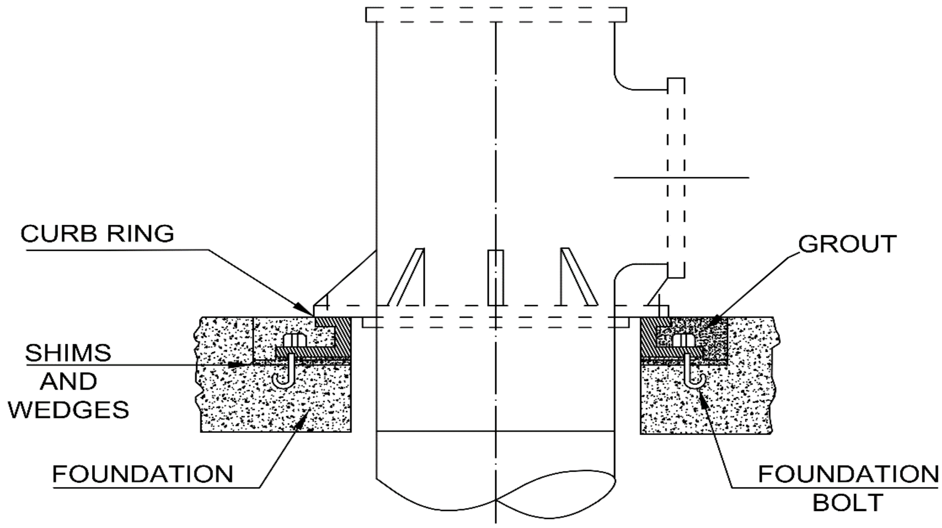
(b) The capacity of the soil or the supporting structure should be adequate to withstand the entire load of the foundation and the dynamic load of the machinery. As mentioned in clauses 6.2.2 and 6.2.3 of IS: 2974 (Part IV), the total load of the pump and the foundation should include the following:

- constructional loads;
- three times the weight of the pump;
- two times the total weight of the motor;
- weight of water in the column pipe;
- half of the weight of the unsupported pipe connected to the pump flanges.

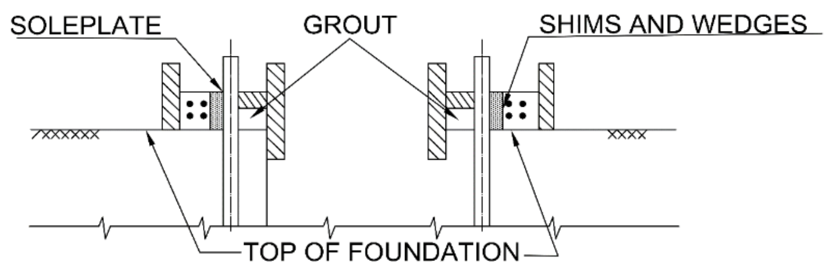
(c) If the pumps are mounted on steel structures, the location of the pump should be as nearest as possible to the main members (i.e., beams or walls). The sections of structures should also have corrosion allowance.

(d) A curb ring or sole plate with a machined top should be used as a bearing surface for the support flange of a vertical turbine pump. The mounting face should be machined because the curb ring or sole plate is used to align the pump. Fig. 5.21 shows a typical arrangement with a curb ring and with a sole plate. A curb ring or sole plate is highly desirable for the VT pump. The sole plate is preferred and permanently installed after blue matching sole plate top surface with discharge head and need not be removed when the pump is lifted for repairs.

(e) Pumps kept in storage for a long time should be thoroughly cleaned and bearings checked, before installation,



(a) ROUND TYPE CURBING FOR ABOVE GROUND DISCHARGE VERTICAL PUMP



(b) GROUTING FORM FOR VERTICAL PUMP SOLEPLATE

Figure 5.21: Foundation for Vertical Pump

(f) Submersible pumps with wet-type motors should be filled with water and the opening should be properly plugged after filling the water.

(g) Alignment of the pump sets should be checked even if they are received aligned by the manufacturers. The alignment should be proper both for parallelism (by filler gauge) and for co-axiality (by straight edge or by dial gauge).

During all alignment checks, both the halves should be pressed hard over to one side while taking the reading.

Alignment should also be checked after fastening the piping and thereafter, periodically during operation.

5.15 Pump Inertia

Normally I, Motor Inertia, is available from motor manufacturers directly and I, Pump Impeller Inertia is available from pump manufacturers. Both of these information can sometimes be obtained from the pump vendor. In case motor and pump inertia are not available, these can be estimated separately and then summed up using an empirical relationship developed by Thorley as given below:

$$I_{pump} = 1.5 \times 10^7 \times \left(\frac{P}{N^3}\right)^{0.9556} \text{ kg m}^2$$

$$I_{motor} = 118 \times \left(\frac{P}{N}\right)^{1.48} \text{ kg m}^2$$

Where

P is the power in kilowatts at the BEP

N is the rotational speed in rpm

5.16 Energy efficiency in Pumps by Flow Control Strategies

5.16.1 Pump control by varying speed

As can be seen from the above laws, doubling the speed of the centrifugal pump will increase the power consumption by eight times. Conversely, a small reduction in speed will result in a drastic reduction in power consumption. This forms the basis for energy conservation in centrifugal pumps with varying flow requirements.

Small increases in the speed of a pump significantly increase power absorbed, shaft stress, and bearing loads. It should be remembered that the pump and motor must be sized for the maximum speed at which the pump set will operate. At higher speed, the noise and vibration from both pump and motor will increase, although for small increases, the change will be small. If the liquid contains abrasive particles, increasing speed will give a corresponding increase in surface wear in the pump and pipework.

Flow control by speed regulation is always more efficient than by control valve. In addition to energy savings, there could be other benefits of lower speed. The hydraulic forces on the impeller, created by the pressure profile inside the pump casing, reduce approximately with the square of speed. These forces are carried by the pump bearings and so reducing speed increases bearing life. It can be shown that for a centrifugal pump, bearing life is inversely proportional to the 7^{th} power of speed. In addition, vibration and noise are reduced and seal life is increased, provided the duty point remains within the allowable operating range.

5.16.2 Pumps in parallel switched to meet demand

Another energy efficient method of flow control, particularly for systems where the static head is a high proportion of the total, is to install two or more pumps to operate in parallel. Variation of flow rate is achieved by switching on and off additional pumps to meet demand. The combined pump curve is obtained by adding the flow rates at a specific head.

The system curve is not affected by the number of pumps that are running. For a system with a combination of static and friction head loss, it is seen that the operating point of the pumps on their performance curves moves to a higher head and, hence, lower flow rate per pump, as more pumps are started. It is also apparent that the flow rate with two pumps running is not double that of a single pump. If the system head were only static, then the flow rate would be proportional to the number of pumps operating.

It is possible to run pumps of different sizes in parallel if the operating head in parallel operation is less than shut-off heads of all pumps and individual discharges of pumps are above the minimum discharge values of the individual models. By arranging different combinations of pumps running together, a larger number of different flow rates can be provided into the system.

Care must be taken when running pumps in parallel to ensure that the operating point of the pump is controlled within the region deemed as acceptable by the manufacturer. It can be seen that if one or two pumps were stopped, then the remaining pump(s) would operate well out along the curve where NPSH is higher and vibration level increased, giving an increased risk of operating problems. While drafting specification, care must be taken to stipulate that the pumps shall be suitable over a specified head range due to varying operating conditions, from a solo operation to parallel operation, up to a specified maximum number of pumps and WL variation from LWL to TWL. All variations in related parameters, i.e., discharge, head, the power drawn and NPSHr should be within design limits and noise and vibration should be within applicable limits.

5.16.3 Stop/Start control

In this control method, the flow is controlled by switching pumps on or off. It is necessary to have a storage capacity in the system, e.g., a wet well, an elevated tank, or an accumulator-type pressure vessel. The storage can provide a steady flow to the system with an intermittent operating pump. When the pump runs, it does so at the chosen (presumably optimum) duty point, and when it is off, there is no energy consumption. If intermittent flow, stop/start operation, and the storage facility are acceptable, this is an effective approach to minimise energy consumption.

The stop/start operation causes additional loads on the power transmission components and increased heating in the motor. The frequency of the stop/start cycle should be within the motor design criteria and checked with the pump manufacturer.

It may also be used to benefit from “off-peak” energy tariffs by arranging the run times during the low tariff periods.

5.16.4 Flow control valve

With this control method, the pump runs continuously and a valve in the pump discharge line is opened or closed to adjust the flow to the required value.

To understand how the flow rate is controlled, see Figures 5.18 and 5.22. With the valve fully open, the pump operates at a higher flow. When the valve is partially closed it introduces an additional friction loss in the system, which is proportional to square of the flow rate. The new system curve cuts the pump curve at lower flow, which is the new operating point. The head difference between the two curves is the pressure drop across the valve.

It is a usual practice with valve control to have the valve 10% shut even at maximum flow. Energy is therefore wasted, overcoming the resistance through the valve at all flow conditions. There is some reduction in pump power absorbed at the lower flow rate, but the flow multiplied by the head drop across the valve is wasted energy. It should also be noted that, while the pump will accommodate changes in its operating point as far as it is able within its performance range, it can be forced to operate high on the curve, where its efficiency is low, and its reliability is affected.

The maintenance cost of control valves can be high, particularly on corrosive and solids-containing liquids. Therefore, the lifetime cost could be unnecessarily high.

5.16.5 Variable Speed Drives (VSDs)/Variable Frequency Drives (VFDs)

Pump speed adjustments provide the most efficient means of controlling pump flow. By reducing pump speed, less energy is imparted to the fluid and less energy needs to be throttled or bypassed. There are two primary methods of reducing pump speed: multiple-speed pump motors and variable speed drives (VSDs).

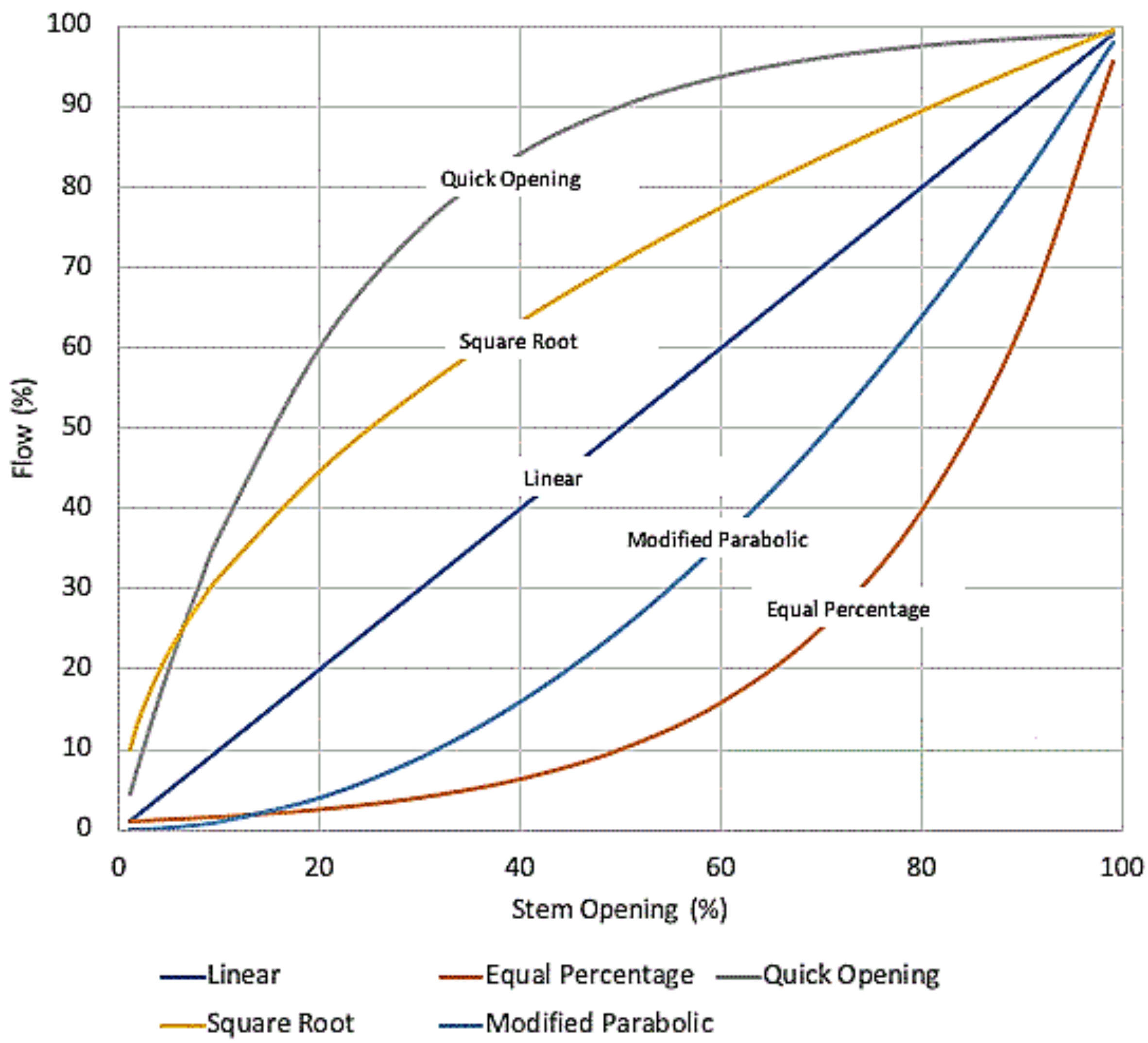


Figure 5.22: Flow Control Valve Characteristics

Although both direct control pump output, multiple-speed motors, and VSDs serve entirely separate applications. Multiple-speed motors contain a different set of windings for each motor speed; consequently, they are more expensive and less efficient than single-speed motors. Multiple-speed motors also lack subtle speed-changing capabilities within discrete speeds.

VSDs allow pump speed adjustments over a continuous range, avoiding the need to jump from speed to speed as with multiple-speed pumps. VSDs control pump speeds using several different types of mechanical and electrical systems. Mechanical VSDs include hydraulic clutches, fluid couplings, and adjustable belts and pulleys. Electrical VSDs include eddy current clutches, wound rotor motor controllers, and variable frequency drives (VFDs). VFDs adjust the electrical frequency of the power supplied to a motor to change the motor's rotational speed. VFDs are by far the most popular type of VSD.

However, pump speed adjustment is not appropriate for all systems. In applications with a high static head, slowing a pump risk inducing vibrations and creating performance problems that are similar to those found when a pump operates against its shut-off head. For systems in which the static head represents a large portion of the total head, caution should be used in deciding whether to use VFDs. VFD manufacturers have to be consulted to avoid the damage that can result when a pump operates too slowly against a high static head. For many systems, VFDs offer a means to improve pump operating efficiency despite changes in operating conditions. When a VFD slows a pump, its head-flow and brake kilowatt curves typically shift downward and to the left, and its efficiency curve shifts to the left. This efficiency response provides an essential cost advantage; by keeping the operating efficiency as high as possible across variations in the system's flow demand, the energy and maintenance costs of the pump can be significantly reduced.

VFDs may offer operating cost reductions by allowing higher pump operating efficiency, but the principal savings derive from the reduction in frictional or bypass flow losses. Using a system perspective to identify areas in which fluid energy is dissipated in non-useful work often reveals opportunities for operating cost reductions.

For example, in many systems, increasing flow through bypass lines does not noticeably impact the back pressure on a pump. Consequently, in these applications, pump efficiency does not necessarily decline during periods of low flow demand. By analysing the entire system, however, the energy lost in pushing fluid through bypass lines and across throttle valves can be identified.

Another system benefit of VFDs is a soft start capability. During start-up, most motors experience in-rush currents that are five to six times higher than normal operating currents. These high currents fade when the motor spins up to normal speed. VFDs allow the motor to be started with a lower start-up current (usually only about 1.5 times the normal operating current). This reduces wear on the motor and its controller.

In most 24×7 distribution systems, VFDs will offer benefits and should be considered at design stage itself.

5.17 Solar Pumps

The solar pump shall conform to specifications prescribed in the notification of the Ministry of New and Renewable Energy, Government of India, New Delhi vide its letter 32/5/2021 - SPV Division dated 08 June 2021.

These specifications cover design qualifications and performance specifications for centrifugal/submersible Solar Photo Voltaic (SPV) water pumping systems from 0.75kW/1 HP up to 11.25kW/15 HP to be installed on a suitable borewell, open well, water reservoir, water stream, etc., and specifies the minimum standards to be followed.in addition to IS 5120 and IEC 62253. These pumps are suitable for emergency use such as power failure on account of floods, cyclones, fires, etc.

Two types of pumps exist, viz., submersible pumps and surface (centrifugal) pumps. Which type of pump is ideal depends on the water source. In the case of a well, the pump needs to be placed underwater. Surface pumps can be placed at the side of a lake or, in the case of a floating pump, on top of the water. Surface pumps are less expensive than submersible pumps, but they are not well suited for suction and can only draw water from about 6.5 metres depth. Surface pumps are excellent for pushing water over long distances.

Solar pumps are powered by solar energy. Solar pumps are inexpensive, long-lasting, simple to install, and require little maintenance. The components of the solar pump are as below:

(a) Solar Panels
(b) Electric motor
(c) Pumps
(d) Inverter
(e) Converter
(f) With battery/Without battery

a) Solar panel

The solar panel consists of photovoltaic modules that generate direct current electricity when exposed to sunlight (Figure 5.23). These panels are hoisted under the open sky supported over steel or masonry structure or over the roof of a building if available. The panels should be installed in shadow free area.

Figure 5.23: Solar Panels

Source: https://www.climateaction.org/news/solar-irrigation-can-improve-prosperity-and-food-security-says-un-agency

b) Electric Motor

The motor is driven by electricity produced by the solar panels exposed to sunlight. The motor may either direct current or alternating current as required.

c) Pump

A pump of required capacity is used either for surface pumping or submersible pumping from borewells.

d) Inverter

An inverter is used to convert direct current electricity into alternating current for use in D.C. or A.C. motors as required.

e) Converter

A converter is an instrument that converts alternating current to direct current, or adjusts the voltage, current, or frequency to help smooth the running of motors.

f) Battery

A battery for 24 hrs. storage capacity is provided to sustain the power supply when the panels are rendered ineffective during clouded sky or rains. Now, submersible pumps are available even without a battery.

5.17.1 Utility of Solar Pump

Solar pumps are useful for providing water supply to small communities in villages located in remote areas where electricity is not available. It may also be useful in gardening of small strips of garden or crop fields.

5.18 High-pressure pumps used in desalination plant

The high-pressure pump is critical to the overall system because it provides the energy required to overcome osmotic pressure in membrane desalination. The high-pressure pump is mainly divided into two categories such as centrifugal pumps and piston pumps. In general, a multistage centrifugal pump is mainly suitable for large-sized desalination plants. A piston pump is mainly suitable for small sized desalination plants. The desalination high-pressure pumps can significantly reduce engineering costs and are widely used in desalination projects.

5.19 Positive Displacement Pumps

Positive displacement pumps operate by trapping a fixed volume of fluid, usually in a cavity, and then forcing that trapped fluid into the discharge pipe. A centrifugal pump transfers the kinetic energy of the motor to the liquid by a spinning impeller. As the impeller rotates, it draws in the fluid causing increased velocity that moves the fluid to the discharge point.

The main differences between centrifugal (rotodynamic) and positive displacement pumps are highlighted in Table 5.7.

Table 5.7: Performance differences between centrifugal (rotodynamic) pumps and positive displacement pumps

Aspects	Centrifugal	Positive Displacement
Working Principle	Impellers pass on velocity from the motor to the liquid which helps move the fluid to the discharge port (produces flow by creating pressure).	Captures a limited volumes of liquid from the suction and forces to the discharge port (produces pressure by creating flow).
Flow Rate vs Pressure	The flow rate changes as the pressure changes.	With a change in pressure, the flow rate remains constant.
Viscosity	Due to frictional losses inside the pump, flow rate rapidly falls with increasing viscosity, even at moderate thickness.	High viscosities are easily managed owing to the internal clearances.
Efficiency	Efficiency peaks at a specific pressure; any variations decrease efficiency dramatically. When run far from the centre of the curve, it does not perform properly and can cause damage and cavitation.	Efficiency is less affected by pressure, but if anything tends to increase as pressure increases. Can be run at any point along their curve without causing harm or reducing efficiency.
Suction Lift	Suction lift is not achievable with standard models; however, self-priming variants are available, and a manometric suction lift is possible with a non-return valve on the suction line.	Create a vacuum on the inlet side, making them capable of creating a suction lift.

Positive displacement pumps are chosen for their ability to handle high viscosity fluids at high pressures and low flows because pressure does not affect their efficiency. While centrifugal pumps are the most common type of pump installed due to their simplicity, positive displacement pumps can handle difficult conditions where centrifugal pumps may fail due to their ability to run at any point on their curve.

Positive displacement pumps are either reciprocating or rotary types.

Since positive displacement pumps do not have a shut-off head like centrifugal pumps, they must not be operated against a closed valve on the discharge side of the pump. When operating against a closed discharge valve, a positive displacement pump will continuously produce flow and build-up pressure till the line bursts or the pump is severely damaged, or both. As a result, a relief or safety valve on the positive displacement pump's discharge side is necessary. The relief valve might be internal or external. Internal relief or safety valves are usually supplied by the pump manufacturer. It is recommended that an external relief valve be installed in the discharge line, with a return line connected back to the suction line or supply tank.

5.20 Selection of Prime Movers

5.20.1 General

With the universal adoption of the alternating current system of electric energy for light and power, the field of application of A.C. motors as prime movers for all drives either in industries or water supply systems are widely used on account of their economy, compactness, ease in operation/maintenance, etc. In the water supply system, Asynchronous A.C. motors are commonly used as a prime mover for the water pumps but the use of synchronous A.C. motors and D.C. motors under circumstances may not be ruled out.

- Asynchronous A.C. motors
- Synchronous A.C. motors
- D.C. motors

Synchronous motors have two types.

a) Induction motors
b) Commutator motors

Induction motors are mostly used in the water supply system. It consists primarily of two major components: (a) rotor and (b) stator. The stator carries a three-phase winding from a three-phase power supply. The rotors of induction motors are of two types (a) squirrel cage rotor and (b) phase wound rotor. The induction motor having a phase wound rotor is known as a slip ring motor. Generally, either squirrel cage motors or slip ring motors are used as a prime mover for pump drive as per the requirement of the load. The squirrel cage motor is used up to 2500 kW load, whereas for higher loads above that, slip ring motors are used.

Synchronous motors merit consideration when large HP, low-speed motors are required. D.C. motors are used occasionally for pumps where only direct current is available as in ships, railways, etc.

5.20.2 Selection Criteria

The type of motor has to be selected considering various criteria such as the constructional features desired, environment conditions, type of duty, etc. Generally, energy efficient motors which are of the highest standard manufactured in India amongst IE2, IE3, and IE4 shall be selected. Improvement in motor efficiency as per IE criteria is continuing. Design and practicing engineers are advised to update about the availability of motors conforming to the highest IE standard and select motors suitably.

5.20.3 Energy Efficient motors

Energy efficiency and sustainability are becoming important topics for all stakeholders globally. Bureau of Indian Standards (BIS) in the IS 12615-2018 for “Line operated three-phase A.C. Motors (IE Code) Efficiency class and Performance specification” clearly mentions the need for the use energy efficient motors and their impact. The Standard defines three levels of efficiencies for low-voltage motors - IE2, IE3, and IE4 - IE4 being the highest efficiency and provide values of performance characteristics and comparison of energy efficient induction motors. In India, IE2 is the mandatory minimum efficiency. However, the standards, Bureau of Energy Efficiency (BEE), and various industry and government entities encourage the use of higher efficiency motors.

Though IS 12615 follows IEC60034-30-1, additional performances are defined in IS 12615 and clearly mentioned so. These include locked rotor torque, locked rotor current, higher variation in voltage and frequency considering Indian grid conditions, etc.

The European Union and many countries all over the world have LV motors with IE3 guaranteed efficiencies which are mandatory and for variable frequency drives, IE2 is minimum. EU's latest Eco-design Regulation (EU) 2019/1781 also stipulates that from July 2023, motors sold in the range of 75kW to 200kW would have to meet IE4 efficiency requirements.

The Return on Investment (ROI) benefits are better when higher efficiency motors are used in a green field project. The life of motors is high at 15 to 30 years and hence, the intermediate replacement of low-efficiency motors with higher efficiency has relatively lower ROI. In lieu of the benefits of using energy efficiency motors on the running cost as well as the lower impact on the environment, it is recommended that motors above 11KW to 200KW be IE4 and smaller motors where the volumes are higher and envisage more manufacturers to participate, a minimum of IE3 efficiency be specified. Typical example and comparison of efficiencies of IE2 to IE4.

Examples of efficiencies:

KW	Pole	IE2	IE3	IE4
22	4	91.6	93	94 .5
30	4	92.3	93.6	94 .9
37	4	92.7	93.9	95.2
11	6	88.7	90.3	92.3
37	6	92.2	93.3	94 .5
22	2	91.3	92.7	94
55	4	93.5	94 .6	95.7

5.20.4 Constructional Features of Induction Motors

Squirrel cage motors are most commonly used. Normally, the starting torque requirement of centrifugal pumps is quite low and squirrel cage motors are therefore suitable.

Slip ring or wound rotor motor to be used where required starting torque is high as in positive displacement pumps or for centrifugal pumps handling sludge.

The slip ring motors are also used when the starting current has to be very low, such as 1.25 times the full load current; such regulatory limits being specified by the power supply authorities.

In addition, the type of mounting is also an important construction feature. Horizontal pumps like split-case centrifugal pumps, end-suction pumps, etc., require horizontal, foot mounted motors which are covered in IS 1231:1974. Vertical turbine pumps which are underwater and have the column pipe and shaft extending to the top require vertical flange-mounted motors, covered under IS 2223:1983. Further details of different types of mountings of rotating electrical machines are available in IS 2253:1974, or its latest edition.

5.20.5 Voltage Ratings

Table 5.8 gives general guidance on the standards voltages and corresponding range of motor ratings.

For motors of ratings 225 KW and above, where high tension (HT) voltages of 3.3 kV, 6.6 KV, and 11 kV can be chosen, the choice could be made by working out relative economics of investment and running costs, taking into consideration the costs of the transformer, motor, switchgear, cables, etc.

Table 5.8: Selection of Motors Based on Supply Voltages

Supply	Voltage	Range of Motor rating in KW	
		Min.	Max.
Single phase A.C.	230 V	0.3	2.5
Three-phase A.C.	415 V	-	250
	3.3 kV	225	750
	6.6 kV	400	-
	11 kV	600	-
D.C	230V	-	150

N.B. When no minimum is given, very small motors are feasible. When no maximum is given very large motors are feasible.

5.20.6 Type of Enclosures:

Table 5.9 gives guidance on the type of enclosures and the place where it is used.

Table 5.9: Types of Enclosures

Type	Environment Code as per IS	Where used
Screen protected drip proof (SPDP)	IP.23	Indoor, clean (dust-free) environment
Totally enclosed fan cooled (TEFC, IC4A1A1)	IP.44	Indoor, dust-prone areas
	IP.54	Normal outdoor
	IP.55	Outdoor at places of heavy rainfall
Totally Enclosed, Self-Water Cooled (TESWC, IC4A0W0)	IP68	Directly submerged under Water (to be pumped)

These days, for motors above 225 kW, HT motors are used for which higher grade of cooling and enclosure protection are required. The types, as under, are stated in increased order of cost and effective cooling:

i) Totally Enclosed, Self-Water Cooled (TESWC)
ii) Totally Enclosed Tube Ventilated (TETV)
iii) Closed Air Circuit Air Cooled (CACA)
iv) Closed Air Circuit Water Cooled (CACW)

HT motor of appropriate enclosure and cooling arrangement from the above three categories shall be selected and further details of enclosure and application are available in IS 13555:1993.

5.21 Class of duty and number of starts

i. All motors should be suitable for continuous duty, i.e., Class S1 as specified IS: 325

ii. Allowable number of starts are as follows:
   - Two consecutive starts from cold condition with second start only after the motor stops fully,
   - One hot restart under high steady state temperature,
   - Permissible number of starts depends on the kW rating, speed, moment of inertia, and stoppage intervals. Generally, for lower kW, higher number of starts per hour are permissible and vice versa. Similarly, the lesser the speed, the greater the number of starts per hour are permissible.

For practical application, the minimum number of starts, as under, can be followed.

Synchronous RPM	Number of starts per hour	Minimum rest (minutes)
3,000	2	20
1,500	3	15
1,000	4	10

5.22 Insulation

Class B insulation is generally satisfactory since it permits temperature rise up to 80 °C. In cool places having ambient up to 30 °C, motors with Class E insulation can also be considered. In hot places having ambient above 40 °C, motors with Class F insulation should be considered. Generally, for hot places, even if Class F insulation is selected, the temperature rise limit is specified as applicable for Class B insulation. If altitude at installation exceeds 1,000 m above mean sea level, the temperature rise limit is reduced to 1 °C per 100 m.

5.23 Starters

5.23.1 Types

Starters are of different types, viz., direct online (DOL), star delta, autotransformer, and stator rotor. Of these, the last one is used with slip ring motors. The other three are used with squirrel cage motors.

5.23.2 Starters for Squirrel Cage Motors

Starters draw starting current, which is considered as a multiple of the full load current (FLC) of the motor. Different types of starters help control the starting current required. General guidelines are given in Table 5.10.

Table 5.10 Guidelines for Starters for Squirrel Cage Motors

Type of Starter	Percentage of voltage reduction	Starting Current	The ratio of starting torque to locked rotor torques, %
DOL	Nil	6 X FLC	100
Star delta	58%	2 X FLC	33
Autotransformer	Tap 50%	1.68 X FLC	25
	Tap 65%	2.7 X FLC	42
	Tap 80%	4 X FLC	64

Note: As per the torque speed characteristics of the motor, the torque of the motor at the chosen percentage of reduced voltage should be adequate to accelerate the pump to the full speed.

5.23.3 Method of Starting

Squirrel cage motors when started directly online (with DOL, starter) draw starting current about six times the full load (FL) current. If the starting current has to be within the regulatory limits specified by the power supply authorities, the squirrel cage motors should be provided with the star delta starter or autotransformer starter.

5.23.4 Selection of the Tapping of Autotransformer type Starter

The torque available from the motor is generally much higher than the starting torque required by the pump, as the starting torque required by the pump is also regulated by starting the pump with the delivery valve closed or open, depending upon the nature of the power versus Q characteristics of the pump.

The torque available from the motor being more than the starting torque required by the pump draws an unnecessary excessive current. This can be controlled by the torque available from the motor, the voltage to be applied to the motor can be reduced by selecting the appropriate percentage by tapping on the autotransformer starter. The value of the percentage for the tapping position can be decided by the following formula.

$$\text{Tapping \%} = 100 \times \sqrt{\frac{Torque\ for\ pump}{Torque\ for\ motor}}$$

Where

Torque for the pump is the torque required to the pump at its rated speed and at its maximum power demand; and

Torque from the motor is the torque available from the motor at its full load capacity and its rated speed at rated voltage.

Based on the above calculation, the nearest higher available position of tapping should be selected.

5.23.5 Reactance Based Starters or Soft Starters:

In the normal start-up of the induction motor, more torque is developed, which causes the stress to be transferred to the mechanical transmission system resulting in excessive wear and failure of the mechanical parts. Soft start offers a dependable and cost-effective solution to these issues by providing a controlled release of power to the motor, resulting in smooth, stepless acceleration and deceleration. Winding and bearing damage are reduced, resulting in a longer motor life. A soft starter is a low-voltage starter for A.C. induction motors.

Soft starters are used on high tension motors due to the following benefits:

i. Smooth starting through torque control for a gradual acceleration of the drive system, preventing jerks and extending mechanical component life.
ii. Reducing starting current to achieve breakaway and holding back current during acceleration to prevent mechanical, electrical, and thermal weakening of electrical equipment such as motors, cables, transformers, and switch gear.
iii. Improved motor starting duty by lowering temperature rise in stator windings and supply transformer.
iv. The microprocessor version of the soft starter has a software-controlled response at full speed that saves energy regardless of load. Because of the tendency to over specify the motor-rated power, this feature has benefits for most installations, not only those where the load is variable.
v. The power factor improvement is a self-monitoring in-built feature. When the motor is running at less than full load, the comparative reactive component of the current drawn by the motor is unnecessarily high due to magnetising and associated losses. As a result of the load proportional active current component, voltage-dependent losses are minimised, and the power factor improves concurrently.
vi. Autotransformer starters provide a lower starting current but take up a lot of switchboard space.
vii. Soft starting and soft stopping minimise the water hammer effect.

The starting performance of the squirrel cage induction motors using soft starters provides valuable economics of electrical energy. Optimum benefits are gained when a motor duty involves frequent start or stop cycles but is still likely to be worthwhile in systems that are in continuous operation.

The disadvantage of soft starter technology over frequency converter technology is that it cannot control speed and is therefore unsuitable for applications that require speed control. The advantage of soft starter technology is that it does not consume power when the motor is in running (unlike a VFD which will always consume power) and does not generate harmonics that may disturb SCADA systems and state electricity grid. It is suitable for constant pumping flow×head applications. It is easily applicable when starting loads with high inertial torque and must be selected at a higher power level.

5.24 Panels

5.24.1 Regulations

The regulations, as per Indian Electricity (IE) Rules for receiving the supply - circuit breaker or switch and fuse units:

(i) For distribution - bus bar, switch fuse units, circuit breakers.

(ii) For controls - starters: level-control, if needed: time-delay relays.

(iii) As protections - under voltage relay, over-current relay, earth fault relay, and single phasing preventer.

(iv) For indications and readings - phasing lamps, voltmeters, ammeter, frequency metre, power factor metre, temperature scanners, indications for the state of the relay, indications for levels indications of valve positions, if valves are power actuated.

The scope and extent of provisions to be made on the panel would depend upon the size and importance of the pumping stations.

5.24.2 Improvement of Power Factor

Power factor is the ratio of KW to kVA drawn by an electrical load, where KW is the actual load power and kVA is the apparent load power. For improvement of power factor, appropriate capacities, operations, and maintenance of the power capacitors are compiled in the following paragraphs. The power factor shall be improved to unity; this shall conform to IS 7752 guides for improvement of power factor.

5.25 Selection of Capacitors

It is generally advisable that capacitors be installed across individual machines. However, in the case of intermittently running machines, it is advisable to select the capacitor of rating appropriate to the average active load for a group of such machines, installing the capacitor across the mains through a fuse switch. A rationalised combination of individual machine mounting of capacitors and a mains installation of capacitors, for a group of machines running intermittently, can also be made in order to maintain a power factor yielding optimum economy. Recommended capacitor ratings are given in Table 5.11.

To have a flexible arrangement for maintaining the power factor within some limits would require an automatic power factor correction panel, monitoring a bank of capacitors for direct connection to induction motors.

Table 5.11: Recommended Capacitor Rating for Direct

Capacitor rating in kVAR when motor speed is							Capacitor rating in kVAR when motor speed is						
Motor kW	3,000 rpm	1,500 rpm	1,000 rpm	750 rpm	600 rpm	500 rpm	Motor kW	3,000 rpm	1,500 rpm	1,000 rpm	750 rpm	600 rpm	500 rpm
2.5	1	1	1.5	2	2.5	2.5	78.3	22	24	27	29	36	41
3.7	2	2	2.5	3.5	4	4	82	23	25	28	30	38	43
5.7	2.5	3	3.5	3.5	5	5.5	85.8	24	26	29	31	39	44
7.5	3	4	4 .5	5.5	6	6.5	89.5	25	27	30	32	40	46
9.3	3.5	4 .5	5	6.5	7.5	8	93.2	26	28	31	33	41	47
11.2	4	5	6	7.5	8.5	9	98	27	29	32	34	43	49
13	4.5	5.5	6.5	8	10	10.5	100.7	28	30	33	35	44	50
15	5	6	7	9	11	12	104.4	29	31	34	36	46	52
16.8	5.5	6.5	8	10	12	13	108	30	32	35	37	47	54
18.7	6	7	9	10.5	13	14.5	112	31	33	36	38	48	55
20.5	6.5	7.5	9.5	11.5	14	16	115.5	32	34	37	39	49	56
22.8	7	8	10	12	15	17	119.3	33	35	38	40	50	57
24.2	7.5	8.5	11	13	16	18	123	34	36	39	41	51	59
26	8	9	11.5	13.5	17	19	126.8	35	37	40	42	53	60
28	8.5	9.5	12	14	18	20	130.5	36	38	41	43	54	61
29.8	9	10	13	15	19	21	134	37	39	42	44	55	62
31.7	9.5	11	14	16	20	22	138	38	40	43	45	56	63
33.6	10	11.5	14.5	16.5	21	23	141.7	38	40	43	45	58	65
35.5	10.5	12	15	17	22	24	145.4	39	41	44	46	59	66
37	11	12.5	16	18	23	25	149.2	40	42	45	47	60	67
41	12	13.5	17	19	24	26	152.9	41	43	46	48	61	68
44.7	13	14.5	18	20	26	28	156.6	42	44	47	49	61	69
48.5	14	15.5	19	21	27	29	160.3	42	44	47	49	62	70
52.2	15	16.5	20	22	28	31	164	43	45	48	50	63	71
57	16	17	21	23	29	32	167.8	44	46	49	51	64	72
59.7	17	19	22	24	30	34	171.5	45	47	50	52	65	73
63.4	18	20	23	25	31	35	175.2	46	48	51	53	65	74
67	19	21	24	26	33	37	180	46	48	51	53	66	75
70.9	20	22	25	27	34	38	182.7	47	49	52	54	67	75
75	21	23	26	28	35	40	185	48	50	53	55	68	76

Connection to Induction Motors (To improve power factor to 0.95 or better)

Note: The recommended capacitor rating given in above Table 5.11 is only for guide purposes. (The capacitor rating should approximately correspond to the apparent power of the motor when it is operating under no-load conditions).

5.25.1 Installation of Capacitors

While installing a capacitor, ensure the following:

(a) A capacitor should be firmly fixed to a base.
(b) Cable lugs of appropriate size should be used.
(c) Two spanners should be used to fasten or loosen capacitor terminals. The lower nut should be held by one spanner and the upper nut should be held by the other to avoid damage to or breakage of terminal bushings and leakage of oil.
(d) To avoid damage to the bushings, a cable gland should always be used, and it should be firmly fixed to the cable entry hole.
(e) The capacitor should always be earthed appropriately at the earthing terminal to avoid accidental leakage of the charge.
(f) There should be a clearance of at least 75 mm on all sides for every capacitor unit to enable cooler running and maximum thermal stability. Ensure good ventilation and avoid proximity to any heat source.
(g) While making a bank, the bus bar connecting the capacitors should never be mounted directly on the capacitor terminals. It should be indirectly connected through flexible leads so that capacitor bushings do not get unduly stressed. This may otherwise result in oil leakage and/or porcelain breakage.
(h) Ensure that the cables, fuses, and switchgear are of adequate rating.

5.25.2 Automatic Power Factor Controller

An APFC panel is used to improve the power factor, whenever needed, by automatically turning on and off the requisite capacitor bank units based on the compensation required in an electrical system.

Power factor is defined as the ratio of active power to apparent power and is an important factor in power conservation.

The power factor controller (PFC) is the command-and-control unit of a capacitor bank system. It switches capacitors to achieve a user-specified target cos ϕ. It is possible to optimise processes, accelerate troubleshooting, and lower the costs of supervised systems by incorporating a PFC.

The aim is to find the amount of reactive power (Qc (kVAR)) that must be installed in order to improve the power factor (cos φ) and decrease the apparent power (S). Qc can be determined from the formula:

$Qc = P (\tan \varphi - \tan \varphi')$.

Where

Qc = power of the capacitor bank in kVAR

P = active power of the load in kW

$\tan \varphi$ = tangent of phase shift angle before compensation

$\tan \varphi'$ = tangent of phase shift angle after compensation

5.26 Transformer

5.26.1 Essential Features

If power requirement exceeds maximum limit of kVA, as per criteria of power supply authority, power supply to the pumping station or any electrical installation is drawn from the power suppliers' grid at a standard grid voltage of 11,000, 33,000, 66,000 volts, etc., depending on the grid voltage. However, the electrical equipment of the consumer will be working at lower voltages like 415 volts, 3,300 volts, or 6,600 volts, depending on the equipment size. This is called consumer side voltage. Transmission of power at such lower voltages will not be economical for the power supplier as this would result in more power losses and necessitate larger conductors for transmission. Hence, the supply voltage is always higher, and the consumer voltage is lower. The power received at the higher voltage is 'stepped down' to a lower voltage by using a power transformer. While the power supply company may supply at a higher voltage and install billing meters at that voltage itself (HT Metering), the consumer or the water utility installs the power transformer to step it down to the required voltage for its use.

The transformer shall conform to IS 2026-2011 of three-phase, copper wound, conventional outdoor type, as per IEC-60076 and IS-1180, with all subsidiary materials like cables, channels, nuts and bolts, air brake switch, etc., as per relevant IS specifications. The transformer shall have complete internal self-protection features (HV fuse, inside HV bushing). A duplicate transformer may be provided, where installation so demands. For a large pumping station and important installation, 1 (Working) + 1 (Standby) transformers shall be provided.

The transformer should be equipped with tap changer to take care of ±10% voltage variation on incoming feeder. Transformer up to 1,000 kVA shall be with manual tap changer in steps of ±2.5% and transformers above 1,000 kVA shall be with on-load tap changer (OLTC) in steps of ±1.25%.

Two types of transformer substations are in use.

- Outdoor substation, where sufficient space is available and generally, majority of substations are outdoor type. Cost of installation is comparatively much less.
- Indoor substation, where problem of space constraint is encountered, or substation is near residential locality. Cost of installation is very high.

5.26.2 Outdoor Substation

a) Pole-mounted transformer, generally for small load up to 63/100 kVA with lightning arrester, air break switch, drop out fuses, insulators, and HT meter.

b) Plinth-mounted transformer substation with insulators, air brake switches, lightning arrestor, bus bar, and HT meter.

5.26.3 Indoor Substations

Indoor substations and UG cabling are provided for ensuring service with minimum breakdowns to overcome the disadvantages of outdoor substations as:

i. Outdoor substations are subject to dust, rain, storm, extreme heat, and theft leading to breakdowns and higher maintenance. During winds, cyclones, and storms, the entire distribution system, including poles and conductors, collapse, taking a long time to restore the power supply.
ii. The indoor substations (Figure 5.24 & 5.25) work at a much lower ambient, say at 28 °C, when the outside temperature may be above 40 °C. Similarly, the UG cable of power distribution is far superior to the overhead system.

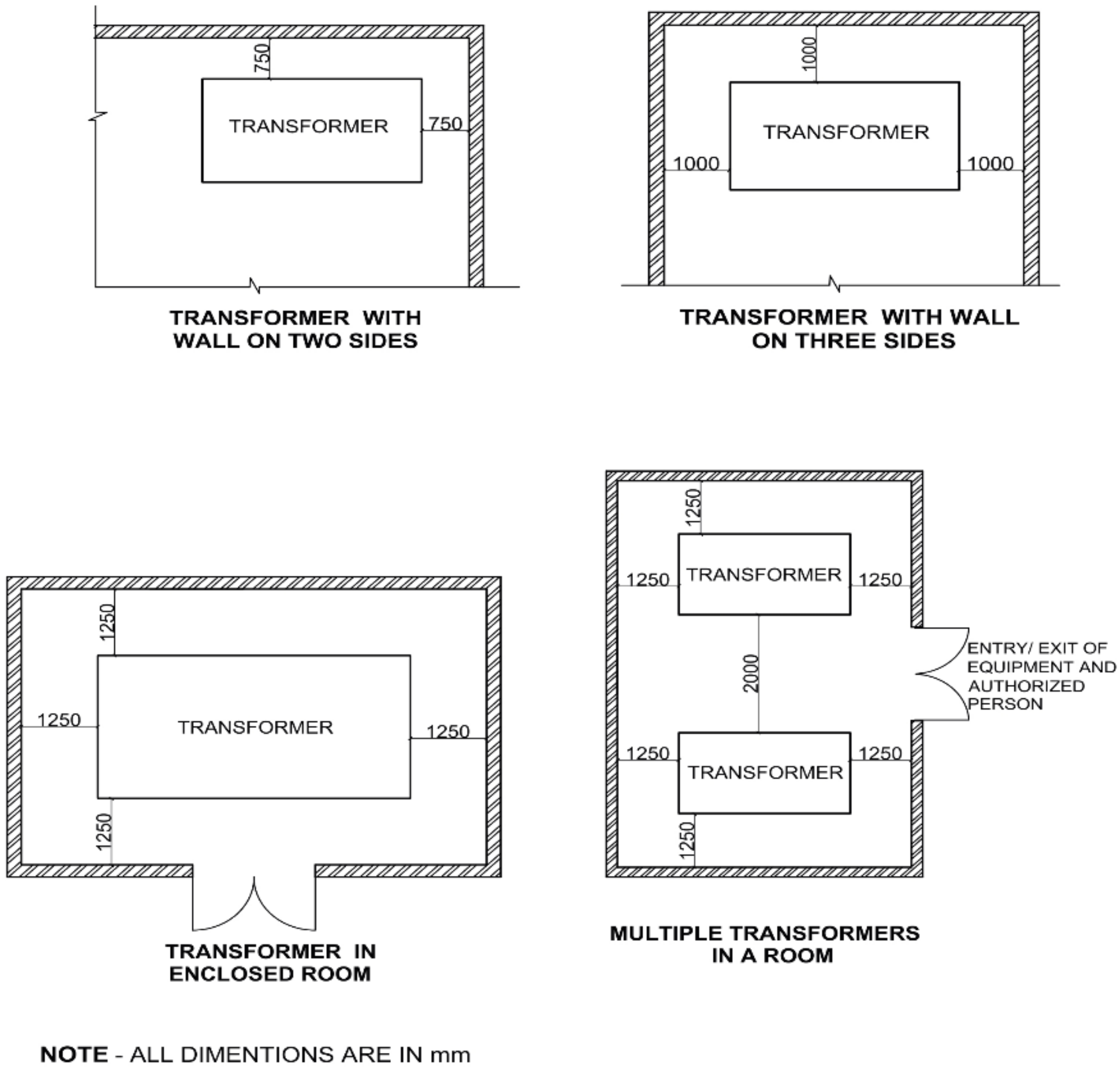
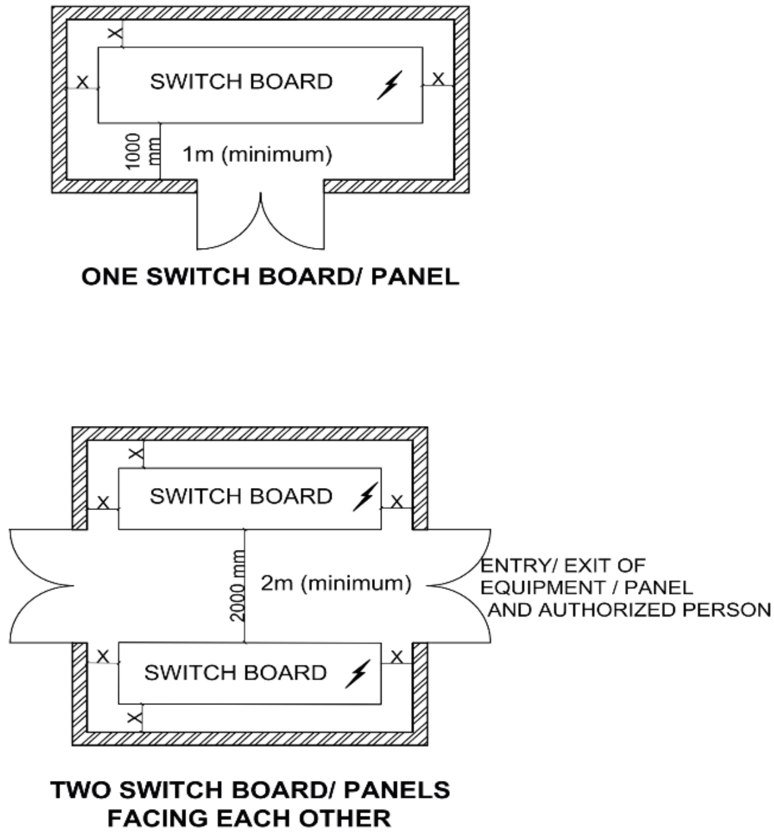


Figure 5.24: Minimum Recommended Spacing between the Transformer Peripheries and Walls

Source: *National Building Code*



'x' is : LESS THAN 200 mm (IF SWITCH BOARD /PANEL IS NOT ACCESSIBLE FROM BEHIND)
: MORE THAN 750 mm (IF SWITCH BOARD /PANEL IS ACCESSIBLE FROM BEHIND)

NOTE - X TO BE MEASURED FROM THE FARTHEST PROTRUDING PART OF ANY ATTACHMENT OR CONDUCTOR.

Figure 5.25: Minimum Recommended Spacing of Switch Board/Panels from Walls

Source: *National Building Code*

5.26.4 Transformer rating

The total power consumption of the pump station should be calculated as below.

a. Power consumption in kW for working motors
b. Power consumption in kW for control equipment
c. Power consumption in cooling, ventilation, lighting, etc.
d. Power factor (PF) 0.85/0.9 to be considered for design purposes
e. Misc. consumption: add 10% of the total a + b + c
f. The total installed capacity shall be at least 15% to 20% higher than the anticipated maximum demand
g. All working pumps except last pump are running and the last pump started. Starting kVA to be considered Momentary kVA under last pump motor starting should not exceed 1.5 times rated kVA of the transformer.
h. A margin for a minimum of one pump motor for future expansion/augmentation is advisable.

5.26.5 Other design consideration

a. With a growing emphasis on energy conservation, the system design is made for both extremes of loading. During the periods of lowest load in the system, it would be desirable to operate only one transformer and to subsequently switch on the additional transformer as the load increases during the day.

b. Total transformer capacity is generally based on present load and possible future load.

c. The selection of the maximum size (capacity) of the transformer is guided by the short circuit making and breaking capacity of the switchgear used in the medium voltage distribution system. Maximum size limit is important from the aspect of feed to the downstream fault.

d. Where two or more transformers are to be installed in a substation to supply a medium voltage distribution system, the distribution system shall be divided into separate sections, each of which shall be normally fed from one transformer.

e. Provisions may, however, be made to interconnect separate sections through a bus coupler in the event of failure or disconnection of one transformer.

5.26.6 Location and Other Requirements

- The substation should preferably be located as near to the load (main pumping station) as possible except for operational clearances. In case of jack well pumping stations, the substation shall be located on the mainland, at a safe height above maximum flood levels, with suitable approaches. In case of an indoor substation, it shall be in a separate building well-ventilated and with natural light, and may be adjacent to the D.G. room, for ease of interconnection.
- All equipment in the substation shall be protected with lightning protection, earthed as per relevant rules and the entire area illuminated at night. The substation should be accessible by vehicle carrying the largest equipment in the station (mostly power transformer). It is also preferable if the substation is visible from the pumping station, as the same operator generally will be manning the substation and pumping station.
- In case there is only one basement in a building, the substation /switch room shall not be provided in the basement. Also, the floor level of the substation shall not be the lowest point of the basement.
- Oil-filled installation - Substations with oil-filled equipment require great consideration for fire detection, protection, and suppression.
- Substations with oil-filled equipment/apparatus (transformers and high voltage panels) shall either be located in an open or in a utility building. They shall not be located on any floor other than the ground floor or the first basement of a utility building. They shall have direct access from outside the building for the operation and maintenance of the equipment.
- Dry-type installation: In case an electric substation has to be located within the main multi-storied building itself for unavoidable reasons, it shall be a dry-type installation with very little combustible material. Such substations shall be located on the ground floor or in the first basement and shall have direct access from the outside of the building for the operation and maintenance of the equipment.
- In the case of two transformers (dry type or transformers with oil quantity less than 2,000 litres) located next to each other without an intermittent wall, the distance between the two shall be a minimum of 1,500 mm for 11 kV, minimum 2,000 mm for 22 kV and minimum 2,500 mm for 33 kV. Beyond 33 kV, two transformers shall be separated by a baffle wall with a 4-hour fire rating.
- The minimum height of the substation/HV switch room/MV switch room shall be arrived at considering the 1,200 mm clearance requirement from the top of the equipment to the bottom of the soffit of the beam.

5.26.7 Generating set

The generator set shall be CPCB-approved, silent type, air cooled, with acoustic enclosures, anti-vibration mountings, foundation, etc., and shall have a standard control panel. The generating set shall be robust in construction, factory tested, and assembled to ensure perfect alignment of engine and alternator on a common base frame. The equipment shall be suitable for operating in a hot humid and saline atmosphere at an ambient temperature of up to 45°C. It should be a multi-cylinder, vertical, four-stroke, direct injection, air/water-cooled type capable of developing the rated horsepower at a speed of 1,500 rpm. The engine shall be with an hour metre to record the hours of operation. The engine shall be started by a completely enclosed axial type of electric starter suitable for 12 volts D.C. The cooling system shall be adequate for the total requirements of the engine when running on continuous full load and on 10% overload for one hour. The exhaust piping system shall be with a residential silencer. The generating set shall have a tank of minimum capacity of 120 litres to enable running of the generator set for 12 hours of continuous run. The base of the genset shall be kept at a minimum of 0.6 m above the ground level so that the oil/fuel can be drained out easily. The insulation shall be Class H. The alternator shall be provided with single bearing or two sleeves to ensure perfect alignment under all conditions. To regulate the generated voltage, a rapid response voltage regulator must be provided. The overall regulations from no load to full load, including cold to hot variation and load power factor of 0.746 lag to unity shall be within 2% of the normal voltage. The sound level shall have less than 75 dB (A) at a distance of 1 metre. The measurement of noise shall be as per ISO 3744/ISO 8528 (Part 10) standard. Typical indoor generator installation in shown in figure 5.26.

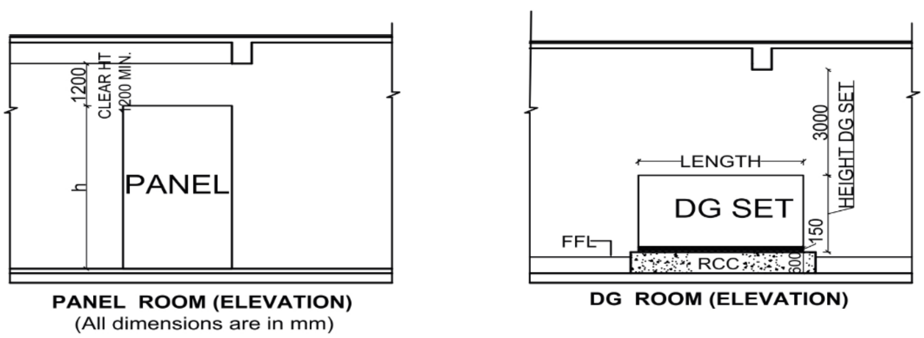


Figure 5.26: Typical indoor generator installation

Source: National Building Code

5.26.8 Generating set rating

The total power consumption of the pump station should be calculated as given below:

a. Power consumption in kW for working motors just before start of last motor

b. Power consumption in kW for control equipment

c. Power consumption in cooling, ventilation, lighting, etc.

d. Misc. consumption - add 10% of the total a + b + c

e. kVA required when last pump set is started considering starting current

f. Generating set kVA = Total kW (a + b + c + d) × Load diversity factor/Power factor × efficiency with last but one pump running + kVA at time of starting

5.26.8.1 Storage for diesel

Adequate facilities for storage of diesel and decanting barrels shall be provided.

5.26.8.2 Low Tension Power Supply (415 Volts)

Where power requirement is less than certain kW, power is taken generally at 415 volts (3 phase) and for very small installation at 230 volts (1 phase). Thus, no transformer is required. In such cases, a voltage stabiliser (1 phase or 3 phase) is provided to correct low or high voltage in incoming power line. The voltage stabiliser shall be of kVA of maximum power load. Setting should be available to improve voltage from 375 volts to 415 volts.

5.27 Cables

Table 5.12 gives guidelines for the types of cables to be used for different voltages.

Table 5.12: Types of Cables for Different Voltages

<table>
<tr><th>Sl. No.</th><th>Range of Voltage</th><th>Type of cable to be used</th><th>Reference</th></tr>
<tr><td>1</td><td>10-230 V or 30-415 V</td><td>PVC insulated; PVC sheathed</td><td>IS 1554</td></tr>
<tr><td rowspan="3">2</td><td rowspan="3">up to 6.6 kV</td><td>PVC insulated; PVC sheathed</td><td>IS 1554</td></tr>
<tr><td>Paper insulated, lead sheathed</td><td>IS 692</td></tr>
<tr><td>XLPE, cross-linked, polyethylene insulated, PVC sheathed</td><td>IS 7098</td></tr>
<tr><td>3</td><td>11 kV</td><td>Paper insulated, lead sheathed, XLPE</td><td>IS 692, IS 7098</td></tr>
</table>

The size of the cable should be so selected that the total drop in voltage, when calculated as the product of current and the resistance of the cable shall not exceed 3%. Values of the resistance of the cable are available from the cable manufacturers.

In selecting the size of the cable, the following points should be considered:

i. The current carrying capacity should be appropriate for the lowest voltage, the lowest power factor, and the worst condition of installation, i.e., duct condition.

ii. The cable should also be suitable for carrying the short circuit current for the duration of the fault.

iii. The duration of the fault should preferably be restricted to 0.1 second by a proper relay setting.

iv. Appropriate for the fault should be applied when cables are laid in a group (paralleled) and/or laid below ground.

v. For laying cables, suitable trenches or racks should be provided.

The three different parameters of cable sizing are given as follows:

- Current carrying capacity
- Voltage regulation
- Short circuit rating

5.27.1 Derating Factors

Cable derating ensures all factors which can increase the temperature experienced by the installation are properly accounted for when selecting cables to prevent cable insulation damage and reduce system losses. The derating factor is used to lower the cable’s current carrying capacity, e.g., if an X-90 cable could carry 40A at 90 °C temperature, additional factors may necessitate derating the cable so that it only carries 30A at 90°C in the installation.

Heat is the main reason why cables need to be derated. Heat is produced as a result of the electrical resistance of the cable as current flows through it. Multiple circuits operating in close proximity can raise the temperature of the conductors due to electromagnetic and physical proximity effects. When cables are arranged close to each other, cables have limited ability to dissipate heat and reach a hotter operating temperature. Linear resistance, or the resistance of the cable per metre, is very small, but it accumulates over a long cable run and causes voltage drop. As the temperature of the cable rises, so does the linear resistance, resulting in increased voltage drop and reduced system output.

5.27.2 Distribution of Water by Direct pumping

Bigger cities require a large number of operational zones and hence, a large number of service tanks. It is a common observation that land is not available for the construction of tanks and hence, in some the cities like Ahmedabad and Chennai, water is distributed by direct pumping.

Smart Pumps

Another reason is that, generally, residual nodal pressures in the existing distribution system are less than 12 m or 17 m as the case may be. In such a situation, direct pumping is proposed. Direct pumping can be through smart pumps. The characteristics of the smart pumps are as follows:

1) Demand-based pumping using smart pumps may be designed for an efficient water distribution network.
2) At the pumping station, the controller should control the pump speed based on the actual flow rate and pressure. To optimise the proportional-pressure curve used by the controller, remote sensors should be installed at critical points in the distribution network, i.e., where a stable pressure is required.
3) The remote sensors should log the pressure throughout the day and send the logged data to the controller as text messages once every 24 hours. Every day the controllers should automatically adapt their proportional-pressure curve, ensuring a stable pressure at the critical points. When the water demand is low, the controller lowers the discharge pressure at the pumping station to save energy and reduce leakages and wear of the pipes.
4) The automatic adoption function should automatically optimise the proportional-pressure curve using the logged pressure data from remote sensors and ensures water is available at a constant pressure at consumers or critical points. The pressure at the pumping station will change depending on the usage at the critical points.

Components - The components may be:

a) The control system should include the pump with variable frequency drive, and other related hardware for 24×7 water distribution. The controller should be of suitable rating, with Modbus RTU on RS485 for SCADA integration.
b) 24×7 system controller must be designed specifically for controlling two to six pumps in water supply pumping stations. The controller can also be integrated into most SCADA systems via a range of different communication protocols embedded in the control hardware. This can also be connected with digital twin technology.

Measures to be taken: Following measures are suggested:

- The cities in which the present water supply is by pumping should prepare GIS maps of the entire pipe network. Condition assessment of the pipes and appurtenances should be shown on GIS maps.
- GIS-based hydraulic model should be prepared.
- Pumps to be used should be of variable frequency drive.
- Exercise for maximum negative pressure (cavitation) of metallic pipes should be made.

5.27.3 Erection and Commissioning

It should be ensured that the direction of the motor agrees with the arrow on the pump. A specimen test should be conducted to derive the system head curve and to understand the actual operating point/range of the pump and the variation, if any, from the original estimated duties. In the case of variations, some analysis may be done to explore any feasible modifications of the system to bring it nearer to the original estimates or to generally improve the system so that it can work better and work trouble-free for long.

Saving Energy in Pumping Stations and Pumping Machinery - A Case Study

Implementation Agency - Oswego Water Department, New York

The City of Oswego Water Department provides potable water to approximately 29,000 customers. The city's conventional water treatment plant has a capacity of 20 million gallons per day (MGD) and an average flow rate of 5-10 MGD. The water system consists of a raw water pumping station, a water treatment plant with a finished water pumping station, three booster pump stations, and water storage tanks with a combined capacity of 11 million gallons.

The city hired an energy performance contractor to provide energy evaluations, energy grant services, and design, bidding, and construction services for the rehabilitation of the raw and finished water pumping stations and booster pump stations. The annual electric cost was approximately $500,000, and the annual natural gas cost was approximately $50,000. Based on contractor recommendations, the following improvements were made:

- rebuilt two 450 horsepower (hp) finished water vertical turbine pumps;
- rebuilt one 350 hp finished water vertical turbine pump;
- replaced motors and variable speed drives at the finished water and raw water pump stations (seven motors from 125-450 HP);
- installed VFDs to modulate pump speeds to maximise energy efficiency;
- installed a SCADA system with remote telemetry;
- upgraded the filter valve actuators;
- upgraded the coagulant chemical feed system; and
- replaced the lighting system.

While improvements cost $2.4 million, the city obtained approximately $270,000 in energy incentives through various NYSERDA programmes. The improvements reduced the peak-electric demand at the facility by 1,463 kW and resulted in an annual electric savings of 1,474,664 kWh and an annual energy cost savings of $95,892. In addition, operation and maintenance savings is approximately $60,000 annually.

(Source: US EPA Strategies for Saving Energy at Public Water Systems)

CHAPTER 6: TRANSMISSION OF WATER

6.1 Introduction

Transmission means the conveyance of water from a source to the water treatment plant (WTP) and thereafter to the distribution system directly or through master balancing reservoir (MBR) or elevated service reservoir (ESR). It includes both raw and clear water transmission. Depending on topography and local conditions, conveyance may be designed for free flow (gravity flow) channels or conduit or pressure conduits. In urban water networks, clear water is normally pumped to an MBR and then conveyed to several ESRs by gravity. This network of pipes that transmits the water without distribution to the consumer is called a *water transmission network*.

There are various types of transmission main systems.

1. Gravity main
2. Pumping main
3. Combined system

6.1.1 Gravity Main

In cases where the source or starting point of the transmission is at a higher elevation and flow in the transmission main occurs from higher potential head to lower potential head, such systems for transmission of water, either open or closed flow is termed as *Gravity System*. A Typical gravity transmission main is shown in Figure 6.1.

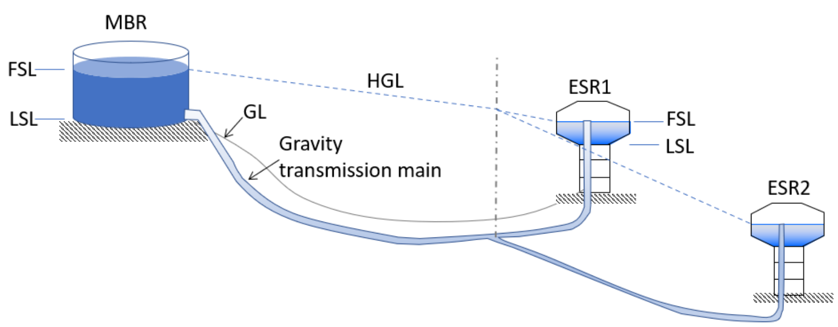


Figure 6.1: Typical Gravity Transmission Main

6.1.2 Pumping Main

When the water has to be transmitted to a higher elevation and starting point of transmission is at a lower elevation, energy/head to the flow has to be provided by an external source. Such a system for transmission of water is termed as *Pumping Main*. A typical pumping main is shown in Figure 6.2. For design of economical diameter please refer to **Annexure 6.1** of Part A of this manual.

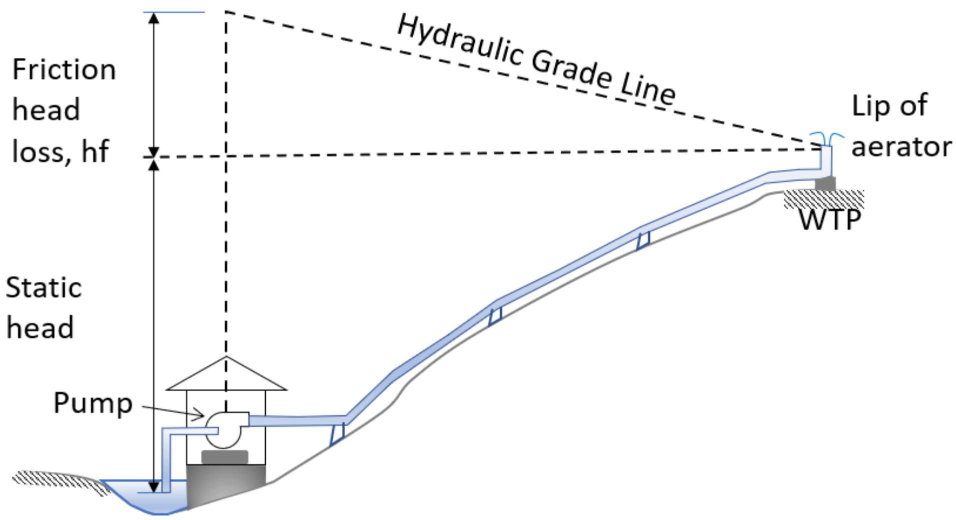


Figure 6.2: A Typical Pumping Main

6.1.3 Combined System

This is the combination of both gravity and pumping mains. Even though sometimes the source/starting point of transmission is at a higher elevation, the advantage of this potential head is not sufficient for the transmission of water. This may be due to friction losses or the presence of a higher elevation enroute to the transmission main. The need arises for providing energy to the system from an external source. Such a system for transmission of water is termed a combined system.

Another case may be when the water is pumped to a nearby higher or similar elevation from where it can be transmitted by gravity main.

The components for transmission of water account for an appreciable part of the capital outlay and hence, careful consideration of the economics is called for, before deciding on the best mode of conveyance.

6.2 Investigation

(i) **GIS:** For marking the GIS drawing of the transmission main, it is necessary to plot the alignment of the pipeline by adding a path in Google Earth and then saving the path as a Keyhole Markup Language Zipped (KMZ) file which is then converted to the shape file using GIS. GIS mapping is extensively discussed in the advisory on “GIS Mapping of Water Supply & Sewerage infrastructure”, dated April 2020, which is available on the website of MoHUA.

(ii) **Topographical Survey**: Topographic survey has to be carried out to essentially cover details such as alignment/route survey with plan and profile along pipeline alignment, and existing structures with locations of temporary benchmarks. In the case of a temporary benchmark, it is necessary to correlate them, and all drawing are brought on a common datum. The simplest way is to use GIS.

(iii) **Geo-technical Investigations**: Geo-technical investigations that will have to be carried out include bore data, bearing capacity for foundation, rock classification, subsoil water table, quality, etc., will have to be carried out. Soil resistivity will have to be carried out to essentially cover details such as resistivity and basic soil survey by taking trial pits along the pipeline alignments.

(iv) **Resistivity Rating**: This factor is important in deciding which of many protective systems to be adopted for buried pipelines.

6.3 Free Flow and Pressure Conduits

6.3.1 Open Channels/Canals

Canals are generally constructed in their economical trapezoidal cross-section whereas rectangular sections prove economical where rock cutting is involved. They may be lined or unlined depending upon the nature of the ground and available slope. Uniform flow occurs in channels where the dimensions of the cross-section, the slope, and the nature of the surface are the same throughout the length of the channel and when the slope is just equal to that required to overcome the friction and other losses at the velocity at which the water is flowing.

Though they are cheap to construct, they are subject to several drawbacks such as loss of water by infiltration/leakage in the ground, evaporation, pollution, seepage, theft, illegal extraction, and deterioration of water quality by the growth of aquatic plants and/or dumping of waste material. Open channels/canals are not recommended for conveying treated water. However, they may be adopted for conveying raw water. Sometimes diversion channels meant for carrying floodwaters from other catchments are also used to augment the yield from the reservoirs.

6.3.2 Flumes

Flumes are open channels constructed in RCC, either supported on the ground or above ground on RCC pillars to transport water over valleys and other depressions in the path of the conduits or along the deep or rocky side of hilly locations.

6.3.3 Gravity Aqueducts and Tunnels

Aqueducts and tunnels are designed such that they flow three-fourth full at the required capacity of supply in most circumstances. For structural reasons, gravity tunnels are generally horseshoe shaped. Gravity flow tunnels are built to conserve the head and reduce the cost of aqueducts, while traversing uneven terrain. They are usually lined to reduce the head loss and reduce seepage. They may be left unlined when they are constructed through stable rock. Mean velocities, ranging from 0.30 to 0.60 m/s for unlined canals and 1 to 2 m/s for lined canals are maintained to reduce eroding of the channels in due course of time.

6.4 Pressure Aqueducts and Tunnels

Pressure aqueducts are generally constructed in RCC. They are generally circular in cross-section and lined. Pressure tunnels are used in large intake work in lakes, reservoirs, and rivers and as the main feeder of distribution systems. Pressure tunnels are constructed to cross rivers and valleys. Normally, the weight of overburden on the tunnels is relied upon to counterbalance the internal pressure. When there is not enough counterbalance to the internal pressure, steel cylinders or other reinforcing structural arrangement needs to be done to provide necessary strength. They share the advantages of gravity aqueduct and additionally they are not exposed to pollution by seepage waters.

6.5 Pipelines and Force Mains

Force mains/rising mains are pressure conduits or pipelines that carry water from the pumping station to the distribution system or from one level to another higher level. Pipelines are pressure conduits of a circular section that generally follow the profile of the ground surface and are laid below the hydraulic grade line.

The materials used in their manufacture/fabrication are cast iron, mild steel, ductile iron, RCC, pre-stressed cement concrete, polyethylene, asbestos cement (AC) pressure pipes, glass reinforced plastic (GRP), and bar wrapped steel cylinder (BWSC).

Details on pipe materials, their classes, PN ratings, design pressures, factory test pressures, field test pressures, available options for external coating and inside lining/painting with merits and demerits, cathodic protection, methods including impressed current method, hydraulic testing of pipeline in the field (Sectional testing as well as complete pipeline testing), laying of the pipeline, beddings, minimum and maximum cover, river crossing, etc., are described in Chapter 11, i.e., “Pipes and Pipe Appurtenances”.

Further, all the valves such as butterfly valves, sluice valves, air valves, valves of cast steel /SG iron, selection of the diameter of air valve vs pipeline diameter, location of line valves, scour valves, air valves, spacing between air valves, air valves with vertical pipe, valves required to be used above 160 m working pressure or 240 m design pressure, are described in Chapter 11, i.e., “Pipes and Pipe Appurtenances”.

6.5.1 Head Loss in Pipes

When a real fluid flows through a pipe, a part of the total energy is utilised in maintaining the flow. This energy is represented in terms of head of water and when it is utilised, it is termed as head loss. The major head loss in the pipe is due to friction and is termed as frictional head loss. There are several minor losses, which are caused due to changes in the magnitude, direction, or distribution of the velocity of flow.

Using the energy principle, Darcy-Weisbach derived a formula to calculate the head loss. This formula requires trial and error or iterative procedure when used in the analysis and design of water distribution networks. To avoid difficulty in using Darcy-Weisbach’s formula, several empirical formulae were developed. However, Hazen-Williams’ formula for pressure conduits and Manning's formula for free flow conduits have been popularly used.

6.5.1.1 Darcy-Weisbach's Formula

Darcy-Weisbach suggested a dimensionless (dimensionally homogeneous) equation for pipeline problems:

$$h = \frac{fLV^2}{2gD} \qquad (6.1)$$

Where, h = Head loss due to friction over length in metres; f = Dimensionless factor; g = Acceleration due to gravity in m/s^2; V = Velocity in m/s; L = Length in metres; D = Diameter in metres

The Colebrook-White formula can be used for calculation of friction factor, f:

$$\frac{1}{\sqrt{f}} = -2log\left[\left(\frac{k}{3.7D}\right) + \frac{2.51}{Re\sqrt{f}}\right] \qquad (6.2)$$

Where, f = Darcy's Friction Factor or Coefficient; R_e= Reynold's Number = (Velocity × diameter)/Viscosity; k = Average height of Roughness projections.

For more details on the Colebrook-White formula, reference may be made to any standard reference book on Fluid Mechanics.

Reference be made to IS: 2951 for calculation of Head Loss due to friction according to Darcy-Weisbach formula.

Recommended design values of roughness projections (k) for pipe materials are shown in Table 6.1.

Table 6.1: Design Values of roughness projections (k)

S. No.	Pipe Material	Value of 'k' mm	
		New	Design
1	Metallic Pipes Unlined - Cast Iron and Ductile Iron	0.15	*
2	Metallic Pipes Lined - Mild Steel	0.06	*
3	Asbestos Cement, Cement Concrete, Cement Mortar or Epoxy lined Steel, CI, and DI pipes	0.035	0.035
4	PVC, GRP HDPE, PVC-O, and other plastic pipes	0.03	0.03

* Reference be made to {IS: 2951 (Part I)} for roughness values of aged metallic pipes.

6.5.1.2 Hazen-Williams Formula

The Hazen-Williams formula is expressed as:

$$V = 0.849C(r^{0.63})(S^{0.54}) \qquad (6.3)$$

Where, V = Average Velocity of flow in m/s; C = Hazen-Williams coefficient; r = Hydraulic mean radius in m; S = Slope of hydraulic grade line (h/L).

For circular conduits of diameter D, the expression for head loss in terms of discharge can be simplified as

$$h = 10.68\left(\frac{Q}{C}\right)^{1.852}\left(\frac{L}{D^{4.87}}\right) \qquad (6.4)$$

Where, L and D are in metres and Q is in cumecs.

6.5.1.3 Manning's Formula

Manning's formula is:

$$V = \left(\frac{1}{n}\right) r^{2/3} S^{1/2} \qquad (6.5)$$

Where, V = velocity of flow in m/s; and n = Manning's coefficient of roughness, r = hydraulic radius (m), S = slope of pipe, m/m)

For a circular conduit of diameter D, the head loss can be written as

$$h = 10.29\,(Q \times n)^2\left(\frac{L}{D^{16/3}}\right) \qquad (6.6)$$

6.5.1.4 Coefficient of Roughness for Different Pipe Materials

In today's economic climate, it is essential that all water utilities ensure that their resources are invested judiciously and, hence, there is an urgent need to avoid over designing of the pipelines.

The coefficient of roughness depends on Reynolds number (hence on velocity and diameter) and relative roughness (k/D). For Reynolds number greater than 10^7, the friction factor 'f' (and hence the C-value) is relatively independent of diameter and velocity. However, for normal ranges of Reynolds number of 4,000 to 10^6, the friction factor 'f' (and hence the C-value) does depend on diameter, velocity, and relative roughness.

PVC, glass reinforced plastic (GRP), and other plastic pipes are inherently smoother compared to AC pressure pipes, concrete and cement mortar/epoxy lined metallic pipes. Depending on the quality of workmanship during manufacture and the manufacturing process, the asbestos cement, concrete, and cement mortar/epoxy lined metallic pipes tend to be as smooth as PVC, GRP, and other plastic pipes.

The metallic pipes lined with cement mortar or epoxy and concrete pipes behave as smooth pipes and have shown C-values ranging from 140 to 145 depending on diameter and velocity.

With a view to reduce corrosion, increase smoothness, and prolong the life of pipe materials, the metallic pipes are being provided with durable smooth internal linings. Concrete, asbestos cement, and cement mortar/epoxy lined metallic pipes, PVC, GRP, and other plastic pipes may not show any significant reduction in their carrying capacity with age and therefore the design roughness coefficient values (C-values) should not be substantially different from those adopted for new pipes.

However, pipes carrying raw water are susceptible to deposition of silt and the development of organic growth resulting in the reduction of the carrying capacity of such pipes. In case of the build-up of substantial growth/build-up of deposits in such pipes, they can be removed by scraping and pigging the pipelines.

Unlined metallic pipes under several field conditions such as carrying waters having a tendency for incrustation and corrosion, low flow velocity and stagnant water, and alternate wet and dry conditions (resulting from intermittent operations), undergo a substantial reduction in their carrying capacity with age. Therefore, lower 'C' values have been recommended for the design of unlined metallic pipes. As such, the use of unlined metallic pipes should be discouraged.

The values of the Hazen-Williams coefficient 'C' for new conduit materials and the values to be adopted for design purposes are shown in Table 6.2. Design purpose ‘C’ values are the same as that of new pipes or lesser. These have been suggested by considering the deterioration of pipe surface over the design period.

Table 6.2: Hazen-Williams Coefficients

Pipe Materials	Recommended C-Values	
	New Pipes@	Design Purpose
Unlined Metallic Pipes		
Cast Iron, Ductile Iron	130	100
Mild Steel	140	100
#Galvanised Iron above 50 mm dia.	120	100
#Galvanised Iron 50 mm dia. and below used for house service connections.	120	55
Centrifugally Lined Metallic Pipes		
Cast Iron, Ductile Iron, and Mild Steel Pipes lined with cement mortar or Epoxy/Polyurethane/three-Layer Polyethylene		
Up to 1,200 mm dia.	140	140
Above 1,200 mm dia.	145	145
Projection Method Cement Mortar Lined Metallic Pipes		
Cast Iron, Ductile Iron, and Mild Steel Pipes	130*	110**
Non-Metallic Pipes		
RCC Spun concrete, Pre-stressed Concrete, Bar Wrapped Cement Concrete Pipe Up to 1,200 mm dia.	140	140
RCC Spun concrete, Pre-stressed Concrete, Bar Wrapped Cement Concrete Pipe Above 1,200 mm dia.	145	145
PVC, GRP, and other plastic pipes like MDPE, HDPE, PVC-O, PVC	150	145

Asbestos Cement pressure pipes	150	140

@ The C-values for new pipes included in Table 6.2 are for determining the acceptability of the surface finish of new pipelines. The user agency may specify that a flow test may be conducted for determining the C-values of laid pipelines.

 # The quality of galvanising should be in accordance with the relevant standards to ensure resistance to corrosion throughout its design life.

**For pipes of diameter 500 mm and above; the range of C-values may be from 90 to 125 for pipes less than 500 mm.*

*** In the absence of specific data, this value is recommended. However, in case authentic field data is available, higher values up to 130 may be adopted.*

The coefficient of roughness for use in Manning's formula for different materials as presented in Table 6.3 may be adopted generally for design purposes unless local experimental results or other considerations warrant the adoption of any other lower value for the coefficient. For general design purposes, however, the value for all sizes may be taken as 0.013 for plastic pipes and 0.015 for other pipes.

Table 6.3: Manning's Coefficient of Roughness

Type of lining	Condition	n
Glazed coating of enamel Timber	In perfect order	0.01
	(a) Plane boards carefully laid	0.014
	(b) Plane Boards inferior workmanship or aged,	0.016
	(c) Non-plane boards carefully laid	0.016
	(d) Non-plane boards inferior workmanship or aged	0.018
Masonry	(a) Neat cement plaster	0.013
	(b) Sand and cement plaster	0.015
	(c) Concrete, Steel trowelled	0.014
	(d) Concrete, wood trowelled	0.015
	(e) Brick in good condition	0.015
	(f) Brick in rough condition	0.017
	(g) Masonry in bad condition	0.020
Stonework	(a) Smooth, dressed ashlar	0.015
	(b) Rubble set in cement	0.017
	(c) Fine, well packed gravel	0.020
Earth	(a) Regular surface in good condition	0.02
	(b) In ordinary condition	0.025
	(c) With stones and weeds	0.03
	(d) In poor condition	0.035
	(e) Partially obstructed with debris or weeds	0.05
Steel, BWSC, PSC	(a) Welded	0.013
	(b) Riveted	0.017
	(c) Slightly tuberculated	0.02
	(d) Cement Mortar lined	0.011
Cast Iron and Ductile Iron	(a) Unlined	0.013
	(b) Cement Mortar lined	0.011
Unlined metallic pipes		0.015
Plastic (smooth)/ MDPE/ HDPE/PVC		0.011

Type of lining	Condition	n
Asbestos Cement		0.012
Glass Fibre Reinforced		0.010

The friction factor values in practice for commonly used pipe materials are given in Table 6.4.

Table 6.4: Recommended Friction Factors* in Darcy-Weisbach Formula

<table>
<tr><th rowspan="2">S. No</th><th rowspan="2">Pipe Material</th><th colspan="2">Diameter(mm)</th><th colspan="2">Friction Factor</th></tr>
<tr><th>From</th><th>To</th><th>New</th><th>For Design Period of 30 years</th></tr>
<tr><td>1.</td><td>R.C.C.</td><td>100</td><td>2000</td><td rowspan="10">0.01 to 0.02</td><td rowspan="7">0.01 to 0.02</td></tr>
<tr><td>2.</td><td>A.C.</td><td>50</td><td>1000</td></tr>
<tr><td>3.</td><td>HDPE/MPDE</td><td>20</td><td>1200</td></tr>
<tr><td>4.</td><td>PVC - U</td><td>20</td><td>630</td></tr>
<tr><td>5.</td><td>PVC - O</td><td>63</td><td>1200</td></tr>
<tr><td>6.</td><td>PVC - C</td><td>15</td><td>150</td></tr>
<tr><td>7.</td><td>Stoneware</td><td>100</td><td>600</td></tr>
<tr><td>8.</td><td>C.I. (for corrosive waters)</td><td>100</td><td>1500</td><td>0.053 to 0.03</td></tr>
<tr><td>9.</td><td>C.I. (for non-corrosive waters)</td><td>100</td><td>1500</td><td>0.034 to 0.07</td></tr>
<tr><td>10.</td><td>Cement Mortar or Epoxy Lined metallic pipes (Cast Iron, Ductile Iron, Steel)</td><td>100</td><td>2000</td><td>0.01 to 0.02</td></tr>
<tr><td>11.</td><td>G.I.</td><td>15</td><td>150</td><td>0.014 to 0.03</td><td>0.315 to 0.06</td></tr>
<tr><td>12.</td><td>PSC</td><td>300</td><td>2600</td><td rowspan="2">0.01 to 0.02</td><td rowspan="2">0.01 to 0.02</td></tr>
<tr><td>13</td><td>BWSC</td><td>250</td><td>1900</td></tr>
</table>

* Values of *f* can also be considered from the Moody's diagram. Reference be made to IS: 2951 for calculation of head loss due to friction according to Darcy-Weisbach formula.

6.5.2 Reduction in Carrying Capacity of Pipes with Age

The carrying capacity of the pipeline depends on the diameter and the Hazen-Williams C-value, which is proportional to the smoothness of the interior surface of the pipe. The higher the C-factor, the smoother the pipe, the greater the carrying capacity, and the smaller the friction or energy losses from water flowing in the pipe. The water carrying capacity of pipes decreases with age due to incrustations (deposition of solids). In effect, the diameter of the pipe and the Hazen-Williams C-value get reduced. The reduction in diameter and C-value causes increase in frictional loss and is reflected in the gradual reduction in carrying capacity of the pipeline and reduction in tail end pressures. So, it can be said that the loss in carrying capacity is caused by: (1) a decrease in the cross-section due to the accumulation of deposits on the interior of the pipes, and (2) an increase in the roughness.

6.5.2.1 Discussion on Various Formulae for Estimation of Frictional Resistance

(i) The Darcy-Weisbach formula is dimensionally consistent. However, its use for the estimation of velocity/discharge during the analysis of the network, or diameter in the design of the network is tedious. As the f value cannot be calculated if velocity or diameter are not known, a repetitive method is required. Initially, f is assumed, and the unknown velocity/discharge/diameter as the case may be is calculated. Then, the calculated value of velocity/discharge/diameter f is obtained using the Colebrook-White formula. If the obtained value is found to be the same, the process is terminated, else the obtained value is considered, and the process is repeated.

(ii) The Hazen-Williams formula is derived for a hydraulic mean radius of 0.3 m and friction slope of 1/1000. However, the formula is used for all ranges of diameter and friction slopes. The formula is dimensionally inconsistent, and the Hazen-Williams C can be considered to have the dimension of $L^{0.37}T^{-1}$, and therefore is dependent on velocity, diameter, and other parameters. However, the Hazen-Williams coefficient C is usually considered independent of pipe diameter, the velocity of flow, and viscosity.

While the DW equation can be used to any Newtonian fluid, the HW formula was created specifically for water. The network's flow is typically turbulent, hence the HW does not deal with laminar flows. It goes without saying that there is virtually no head loss at that low velocity. The answers of the HW and DW equations coincide for a certain Reynolds number. The outcomes somewhat deviate as one goes from that value. The impact is particularly noticeable on rough pipes. However, for smooth pipes, the changes are typically negligible. In cases when pipes with a diameter of 1800 mm or more have exceptionally high Reynolds numbers, it can be necessary to lower the C-factor. The viscosity impact of temperature cannot be readily adjusted. Despite all of these distinctions, they are negligible for ordinary water and sewer operations. For over a century, engineers have been designing millions of kilometres of pipes using the HW equation, and those pipes are still in operation today. It is possible to calibrate models created using the HW equation to match actual piping systems.

(iii) If there is a choice for use of pipe friction formulae, Darcy-Weisbach which yields accurate results can be preferred over the Hazen-Williams (HW) formula. However, no other formula for head loss in pressurised pipe flow conditions should be used.

(iv) Manning's formula is recommended for flow under atmospheric pressure such as in open channels, and partially filled pipes.

6.5.2.2 Method of Determining Value of 'C' for Existing Pipes at Site

Commercial pipes are available in different lengths for different pipe materials. The C-values of individual pipes can be determined in the lab. However, this may not give a correct representation of the C-value of pipes in the field, where pipes are joined in series from one node to the other node. These joints greatly affect the C-value of pipe and therefore, it is sometimes desirable to determine the C-value at the site. The following method can be adopted.

Choose a pipe of the required size of any material for which C-value is required (preferably 100 mm flanged pipe for ease in transportation), transport at a wash water outlet of the existing water supply system, connect with wash water sluice valve flange, tighten the flange of pipe putting rubber insertion between sluice valve flange and pipe flange with nuts and bolts to avoid any leakage. Lay over ground this 100 mm flange pipe at least 105 m in length. Put distinguishable marks 100 m apart on the pipe. The inverted water manometer is accurate and gives a difference of heads up to 1 mm. Hence, it is installed at two marked points 100 m apart on the pipe. Fit ultrasonic flowmeter in between the marks (preferably in the middle). Now, open the wash water valve of the existing water supply to permit water flow. Let the water flow for 5 to 10 minutes and then take at least 10 readings of heads in the manometer at both the marked points and flow rates. Find the density of water by hydrometer by taking five samples of water collected from the outlet of the laid pipe and take five readings. By averaging all the readings, let the following average readings be obtained.

- Average Pressure (first mark (P_1))
- Average Pressure (second mark 100m apart (P_2))
- Average Discharge (flow rate Q)

- Average Density of water (ρ)
- Length of pipe (L)
- Diameter of pipe (D)
- Acceleration due to gravity (g)
- Now, loss of pressure in length 100 m = P_1 - P_2 = P
- Loss of head (h) = P/ρg

Hazen-Williams' formula as in Eq. (6.4) can be used to obtain the C-value of pipe.

Change in 'C' with age can also be determined analytically using the relation given by Sharp and Walski (1988):

$$C = 18.0 - 37.2\, log\left(\frac{\varepsilon_0 + at}{D}\right)$$

Where:

ε_o = roughness height when pipe was new (t=0) (mm)
a = rate of change in roughness height (mm/year)
t = age of pipe (years)
D = diameter (mm)

The corrosivity of the water causing change in roughness height is related using the Langelier Index, shown in Table 6.5.

Table 6.5: Correlation between Langelier Index and the Roughness Growth Rate

Description	a (mm/year)	Langelier Index
Slight attack	0.025	0.0
Moderate attack	0.076	-1.3
Appreciable attack	0.25	-2.6
Severe attack	0.76	-3.9

The relationship between C and age is related to the base 10 log of the roughness height and diameter.

6.5.3 Minor head loss due to Specials and Appurtenances

Pipeline transitions and appurtenances add to the head loss, which is expressed either in terms of velocity head as

$$h_m = \frac{KV^2}{2g} \tag{6.7}$$

Where V is the average velocity before the minor loss element, and K is the minor loss coefficient that remains practically constant for high Reynolds' number.

The values of K to be adopted for some typical fittings are given in Table 6.6. Hydraulic tables or standard textbooks and reference books or a manufacturer's catalogue can be used for other special fittings.

Table 6.6: K-Values for Different Fittings/valves

Type of Fittings	Value of K
Sudden contractions/expansion	0.3*- 0.5

Type of Fittings	Value of K
Concentric/Eccentric reducer and enlarger	0.15-0.25
Bellmouth	0.1
Entrance shape well rounded	0.5
Elbow/Bend 90°	0.5-1.0#
45°	0.4-0.75#
22.5°	0.25-0.50#
Tee 90° take-off	1.5
Radial tee	0.8
30/45 degrees tee	1.0
Straight run	0.3
Coupling/Flange adapter/Dismantling joint	0.3
Gate valve/Sluice valve/Knife gate valve (in fully open condition)	0.3-0.4
Globe	10.0
Angle	5.0
Swing check valve/non-return valve/Reflux valve/Dual plate check valve	2.5
Butterfly valve	0.4
Venturi Meter	0.3
Orifice	1.0
Magnetic/Ultrasonic flowmeter	0.1
Discharge head elbow(bend)/Subsurface delivery tee for VT pump	0.5
Foot valve	2.0
Strainer	1.5

* *Varying with area ratios.*

Lowest values are for long radius elbows and highest values are for short radius elbows.

The minor losses in pipes can also be considered through the equivalent length of straight pipe that can be added to the length of the pipe. The equivalent length values of pipe for different sizes of various fittings with K=1 is given in Table 6.7.

Table 6.7: Equivalent Length of Pipe for Different Sizes of Fittings with K = 1

Size in mm	Equivalent length of pipe in metres		Size in mm	Equivalent length of pipe in metres
10	0.3		65	2.4
15	0.6		80	3
20	0.75		90	3.6
25	0.9		100	4.2
32	1.2		125	5.1
40	1.5		150	6
50	2.1			

6.6 Guidelines for Cost-Effective Design of Pipelines

The cost of the transmission and distribution system constitutes a major portion of the project cost. It is desirable to adopt the following guidelines:

(i) In the design of distribution systems, the minimum design velocity should be selected in such a fashion to avoid the deposition at the bottom of the pipe which may result in deterioration of pipe quality. A minimum velocity of 0.4 to 0.6 m/s is recommended to avoid depositions and consequent loss of carrying capacity. However, where inevitable due to minimum pipe diameter criteria or other hydraulic constraints, lower velocities up to 0.3 m/s may be adopted with adequate provision for scouring.

(ii) The maximum flow velocity should not be more than *2.5* m/s for raw water to avoid the abrasion and subsequent scouring in the pipelines due to suspended particles. However, in case of filtered water, as the quantity of solids (which contribute to the abrasion) is negligible, the maximum flow velocity to be adopted shall be 3 m/s.

(iii) For hilly area and branch pipe connecting transmission main to service reservoir:

The maximum velocity for MS/DI pipes with internal mortar lining shall be limited to 4.0 m/s for following two cases:

a) For hilly regions

b) For part of branch pipe connecting transmission main to service reservoir required for dissipation of excess residual head

(iv) In all hydraulic calculations, the actual internal diameter of the pipe shall be considered after accounting for the thickness of the lining, if any, instead of the nominal diameter or outside diameters (OD).

(v) The Head Loss gradient should not exceed 10m/km. The value of head loss gradient can be exceeded in hilly areas, however, the velocity should not exceed the permissible value of 2.5 m/s.

(vi) It is desirable that head loss due to fittings, specials, and other appurtenances are obtained. However, accounting for an individual head loss of each valve and fitting used in transmission mains and water distribution networks (WDN) is not practically possible. Usually, these minor losses are considered as 10% of the frictional losses. In some of the software that are used for the simulation and design of WDNs, there is no provision for a direct increase in friction loss by a certain percentage. Therefore, either the length, flow, or C-value can be modified appropriately. To account for 10% of minor losses, the length of pipes can be increased by 10% or nodal demand can be increased by 5.28%, or the C-value can be reduced by approximately 5%.

6.7 Economical Size of Transmission Main

6.7.1 General Considerations

When the source is separated by a long distance from the area of consumption, the conveyance of the water over the distance involves the provision of a pressure pipeline or a free flow conduit entailing an appreciable capital outlay. The most economical arrangement for the conveyance is therefore of importance.

The available fall from the source to the town and the ground profile in between should generally help to decide if a free flow conduit is feasible. Once this is decided, the material of the conduit is to be selected, keeping in view the local costs and the nature of the terrain to be traversed. Even when a fall is available, a pumping or force main independently or in combination with gravity main could also be considered. Optimisation techniques need to be adopted to help decisions.

The diameter (D in m) of a free flow conduit connected between two reservoirs having a head difference of h m to carry a known discharge of Q m^3/s can be simply obtained by using the HW head loss formula (Eq. 6.4). This will result in a non-commercial size that can be changed to the next available higher size.

However, the design of a pumping main requires consideration of both pipe size and pump capacity. A smaller pipe size provides the lower pipe cost, however, results in higher head loss and thereby higher pump capacity and higher energy cost. On the contrary, higher pipe size increases the pipe cost, however, due to lesser head loss, both pump capacity and energy charges are reduced. The optimal diameter is the size that minimises the overall cost of pipeline and pump cost and energy cost. Such a diameter may be theoretical and may not be available. Thus, size from the set of available commercial pipe sizes is chosen to minimise the overall cost and is called as *Economical Diameter*. As different types of costs at different times are involved, the theory of economic analysis is used for the comparison of alternatives.

The most economical size for the conveyance main will be based on a proper analysis of the following factors:

(i) The period of design considered is 30 years or the period of loan repayment if it is greater than the design period for the project and the quantities to be conveyed during different phases of such period.
(ii) The different pipe sizes against different hydraulic slopes/acceptable velocity ranges can be considered for the quantity to be conveyed.
(iii) The different pipe materials which can be used for the purpose and their relative costs as laid in position.
(iv) The duty, capacity, and installed cost of the pump sets required against the corresponding sizes of the pipelines under consideration.
(v) The recurring costs on:

a. Energy charges for running the pump sets. Escalation in costs per year also needs to be considered. Usually, the escalation/inflation rate per year is 2% less than the rate of interest,
b. Staff for the operation of the pump sets,
c. Cost of repairs and renewals of the pump sets,
d. Cost of miscellaneous consumable stores, and
e. Cost of replacement of the pump sets installed to meet the immediate requirements, by new sets at an intermediate stage of the design period. The full design period or the repayment period may be 30 years or more while the pump sets are designed to serve a period of 15 years.

6.7.2 Evaluation of Comparable Factors

Every alternative, when analysed on the above lines, could be evaluated in terms of cost figures on a common comparable basis by:

(i) The capital cost of the most suitable pipe material as laid and jointed and ready for service, including the cost of valves and fittings and all ancillaries to the pipeline.

(ii) (a) Capital cost, as installed, of the necessary pump sets corresponding to the pipeline size in (i) above.

(b) The amount which should be invested at present would yield compound interest, the amount necessary to replace the pump sets in (ii)(a) at the end of their useful life with bigger pump sets for once or often to cater to the requirements during the design period or the loan repayment period.

(iii) Energy charges - if the pump sets in (ii)(a) are designed to serve for, say 15 years, the daily pumpage will vary from the initial requirements to the intermediate demand after 15 years. The energy charges will be based on the average of these two daily pumpages, leading to an average annual expenditure on energy charges on such a basis.

The replacing of pumps under (ii)(b) will, likewise, involve annual recurring energy charges for the average of the demands during the subsequent 15 years period for the project design or the loan repayment period whichever is greater.

The two annual recurring costs should be capitalised for inclusion as a part of the present investment. For this purpose, it is necessary to derive:

(a) the amount of the present investment which would yield an annuity for 15 years equal to the annual energy charges on the initial pump sets;
(b) the amount of present investment which would commence to yield, over the subsequent 15 years period, the annual energy charges for the replaced pump sets in (ii)(b);
(c) apart from the energy charges, the other recurring annual charges comprise the cost of operation and maintenance staff, ordinary repairs, and miscellaneous consumable stores.

The present investment which would yield an annuity equal to such annual recurring charges throughout the design period, or loan repayment period (if it exceeds the former), would represent the capitalised cost, for inclusion as part of the total investment now required.

(iv) The addition of the present investment figures as worked out under (i), (ii)(a), (ii)(b), (iii), and (iv) would represent the total capital investment called for in respect of each alternative involving a specific pipeline size and the corresponding pump sets. A comparison of the total investment so required in respect of the several alternatives examined would indicate the most economical pipeline size to be adopted for any project.

(v) In all the above computations, the rate of interest plays an important role and for a proper comparison, it may be taken as the rate demanded for the loan repayment. Also, inflation should be considered and the minimum attractive rate of return, i_r (MAAR) can be obtained by subtracting the inflation rate, i_{in} from the effective interest rate, i_f.

A typical variation of the total cost curve with respect to diameter is shown in Figure 6.3. The curve is a unimodal convex. Therefore, to avoid consideration of all available sizes, few candidate pipe sizes can be selected. This will reduce computational efforts. In case, the economical size is obtained as the lowest or largest from the list of candidate diameters, the process can be repeated by including one of the higher/lower sizes depending on the obtained size. If no higher/lower size is available, the last pipe is the economical size. The number of candidate sizes can be chosen using velocity or hydraulic gradient criteria or using Lea's approximate formula. Lea suggested that the economical diameter in metre usually lies between 0.97 to 1.22 $\sqrt{Q}$, where Q is the design discharge in the pumping main in m^3/s. Thus, four to five commercial diameters in the above range can be selected as candidate diameters.

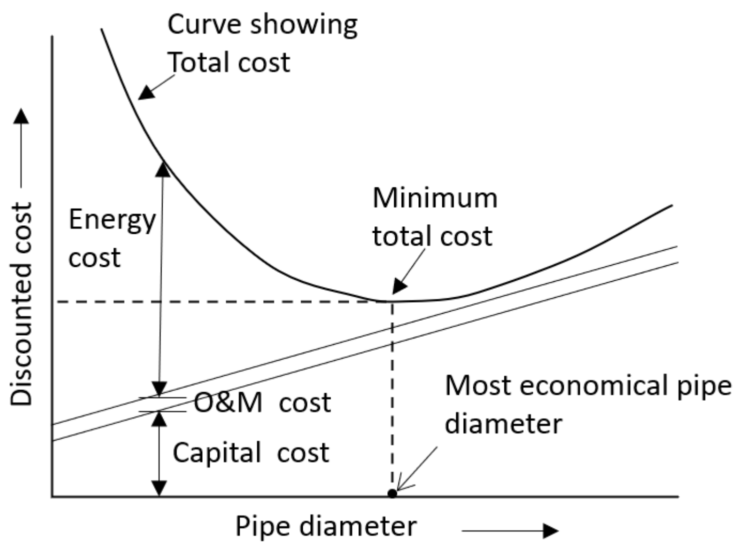


Figure 6.3*:* Variation of Total Cost with Pipe diameter

The stepwise methodology is given and illustrated with an example in **Annexure 6.1**

6.7.3 Scope of Sinking Fund

In the methods of comparison outlined above, any provision for a sinking fund to replace the pipeline or the pump sets at the end of the design or loan repayment period, where needed, has been advisedly not included. It would be tantamount to the present generation paying in advance for the amenities for the next generation, in addition to paying for its own amenities through the design period of 30 years. Such a procedure is neither equitable nor expedient, particularly when local finances are unable to shoulder the financial commitments even against the initial installations of such projects.

6.7.4 Pipeline Cost under Different Alternatives

There are three independent factors bearing on the problem, viz., the design period of 30 years, the loan repayment period, and the life of the pipeline. There is a particular pipe size for which cost should be minimum, considering its capital and maintenance charge, for the loan repayment period. The size of the pipe will be larger if the period considered is the life of the pipeline and this larger size would appear to be less economical if the period is restricted to the loan repayment period.

The issue, therefore, hinges on which size to choose out of the two in a particular project. Whichever size is adopted, the loan, therefore, has to be repaid within the specified period, long before the pipeline ceases to be of use. For the investor, the pipe size which will cost him/her the minimum is the criterion, pipe costs, and maintenance being considered over the loan repayment period. The other size based on the life of the pipe material would cost him/her an additional financial burden although it may be the cheapest when considered over the life period of the pipeline. For the purpose of finding economical diameter, adopting the price as per relevant DSR is good enough.

6.7.5 Life of Pipes

'Pipe Life' is the expected 'Design Useful Service Life' (DUSL) for a particular 'Pipe Material'. The life period of the pipeline will depend on several factors which are as follows:

a) Pipe material and thickness
b) Working pressure of the pipeline

c) Workmanship

d) Operation and maintenance

e) Characteristics of water

f) Surrounding environment

6.7.6 Recurring Charges-Design Period vs. Perpetuity

The annual recurring charges for energy and operation and maintenance are perpetual, irrespective of the design period or the life of the pipeline. Their capitalised value is restricted to the design period or the loan repayment period whichever is greater, as it reflects the commitment involved relevant to such period for a proper comparison between alternatives. Otherwise, a possible method may be considered as an initial investment that would yield interest to meet such recurring charges in perpetuity. It is, however, simple and more rational to consider capitalisation of the recurring charges over the design period for the purpose of designing the diameters.

6.7.7 Capitalisation Vs Annuity Methods

In Section 6.7.2(v), the comparison suggested was based on the present capitalised value. Alternatively, the capital installation cost of the pipeline could be converted into an annuity for the design period, or loan repayment period, whichever is greater, in the same way as a loan discharged through annuities. This annuity can then be added on to the other annual recurring charges for a total comparison between the alternatives.

6.7.8 Selection Principles

The above method suggested for evaluation of comparable factors would give a comparative idea of the total capital investment involved whereas the capitalisation vs. annuity methods would indicate the annuities involved as between the alternatives. A better concept is perhaps afforded by the former method, i.e., capitalisation.

The most economical size of a main can be arrived by evaluating the capital and the operation and maintenance cost (capitalised value for design period of 30 years) for different diameters. Mathematical solution is also possible (Annexure 6.1). The objective (cost) function is formulated to ensure desired system performance. Several optimisation techniques are available for minimising the objective function. One of the simpler methods is one in which its (objective function) first partial derivatives with respect to the several decision variables are set equal to zero. The resulting system of equations is solved exactly or approximately and the principal minors of the determinant of second partial derivatives are investigated to ascertain whether a maximum or minimum is involved.

While determining the type of the pipe material to be used, alternative alignments, cost of cross drainage works, cost of valves, specials, and other appurtenances, should all be considered to determine the most economical size for the conveying main.

6.7.9 L-Section

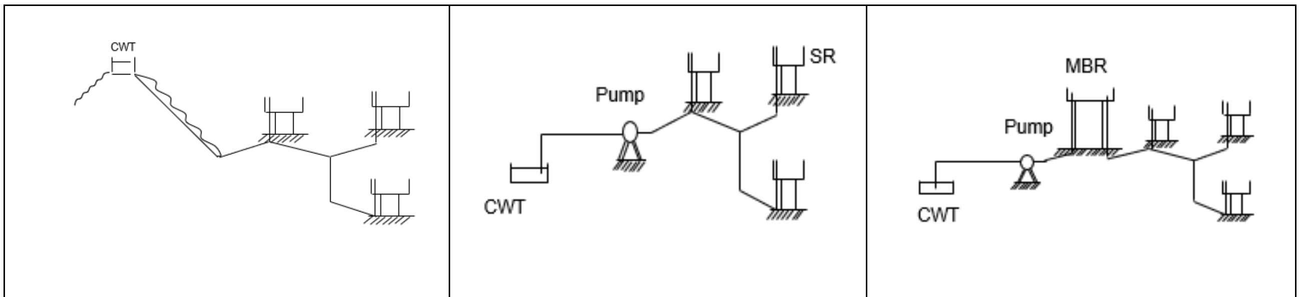
A longitudinal section (L-Section) along the pipeline route must be made to show proper alignment and hydraulic grade after a detailed survey before designing the pipeline, and it is also needed to access the requirements and locations of air valves, scour valves, etc. The L-Section also helps in planning and laying the pipeline and identifying any obstructions and permissions required.

Soil investigation along the alignment to examine the resistivity and corresponding corrosion of soil encountered. Refer to Chapter 11: Pipes and Pipe Appurtenances of Part A Manual.

6.8 Types of Branched Transmission Mains

The economic size design of the pumping main may be said to be a balance between the sizing of the main and the least life cycle cost investment of the system wherein cost of pipes, cost of pump sets, capitalised cost of energy, capitalised cost of operation and maintenance, etc., are considered comparatively for various available sizes of pipes. The pumping main or conveyance main transports water from one location to another location and is not permitted to be tapped between the point of propulsion and the point of reception. However, there could be a direct pumping system feeding to several reservoirs through a network of pipes, or a combined gravity and pumping system in which water from a clear water tank (CWT) at WTP is pumped to an MBR, which in turn supplies to various service reservoirs by gravity. Wherever topology permits, water from the WTP can also be supplied to various reservoirs completely by gravity also.

A typical complete gravity, direct pumping, and combined gravity and pumping system are shown in Figure 6.4 (a), (b) and (c).



(a) CWT - Clear Water Tank at WTP	(b) SR - Service Reservoir	(c)MBR - Master Balancing Reservoir

Figure 6.4: (a) Complete Gravity (b) Direct Pumping (c) Combined Gravity and Pumping

The layout of the transmission main system has great importance on the cost of the network. The layout of a distribution network depends on the existing pattern of streets and highways, existing and planned sub-division of the service area, property right-of-way, possible sites for ground and ESRs, and location and density of demand centres.

Pipes, being lifelines, should be laid along the roads. A minimum spanning tree or shortest path tree from CWT to various ESRs can reduce the cost substantially and should be preferred. Grouping high-level and low-level ESRs in the city should be done, preferably by the use of the GIS technique of the inverse distance weighted (IDW) surface. However, duplication of the pipeline, i.e., parallel pipelines, should be avoided. If necessary, alternatives for layouts can be considered and the one providing the least cost can be selected.

The topography of the service area may be flat or uneven. In an uneven terrain, booster pumps may be necessary for pumping water to high areas within the network. Similarly, it may be necessary to provide pressure-reducing valves for areas with lower elevation to reduce pressure. Check valves (non-return valves) may also be necessary to maintain flow in the selected direction and restrict flow from the opposite direction. The transmission main systems are used for supplying water to various service reservoirs in the city. They are also used in group water supply schemes, in which several villages or a combination of urban towns and villages are supplied from a common source and WTP facilities.

The supply from CWT/MBR to various village/town reservoirs may be direct as shown in Figure 6.5 (a). Such systems may be termed as single level systems. Sometimes, MBR may supply to several zonal balancing reservoirs (ZBRs) which in turn may supply to several village reservoirs (VRs) as shown in Figure 6.5 (b). Such systems may be termed as multi-level systems.

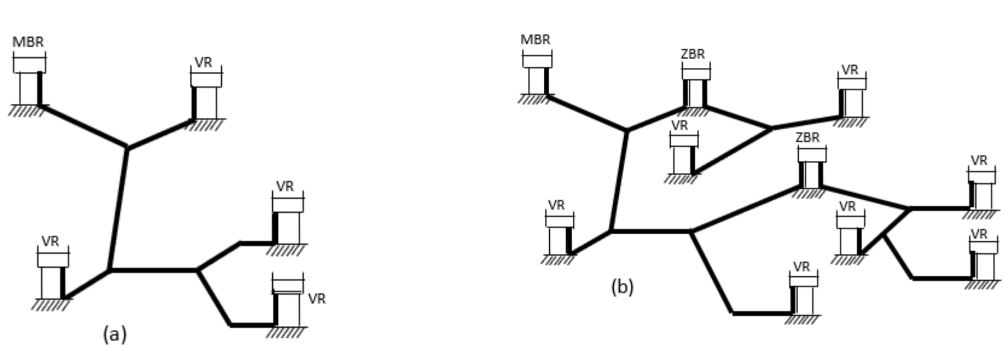


Figure 6.5: Water Transmission System: (a) Single Level; (b) Multi-Level

Note: MBR - Main Balancing Reservoir; ZBR - Zonal Balancing Reservoir; VR - Village Reservoir

6.8.1 Optimisation of Branched Transmission Mains

Several methods for the optimal design of branched networks are available. The methods of the Linear Programming based model and the hydraulic model are discussed here.

(a) Linear Programming (LP) based model: Linear Programming (LP) based model is most useful for the design of branched networks as it provides a global optimal solution considering discrete pipe sizes. The use of Integer Linear Programming (ILP) will avoid the selection of two sizes. BRANCH software, based on LP and JALTANTRA based on ILP, can be used for the optimal design of the transmission main network. Several metaheuristic techniques like Genetic Algorithm, Simulated Annealing, Cross Entropy Optimisation, Particle Swarm Optimisation, etc. have been tested by researchers to obtain an optimal solution and can be used. But presently, some of these software are costly and the same is still not giving truly optimised solutions. Moreover, this software is non-spatial and hence, difficult to manage on the GIS platform.

Using JalTantra Software: “JalTantra” is a freeware system for the optimal design of branched water distribution networks, developed by CSE IIT Bombay. The user has to log in to the website (https://www.cse.iitb.ac.in/jaltantra) to access the JalTantra. JalTantra can be used for all types of water transmission mains (WTN), i.e., gravity, pumping, and combined pumping and gravity networks. In the case of a combined pumping and gravity WTN, JalTantra allows the sizing of pumping main, pump, ESR, and gravity mains simultaneously instead of considering them separately. JalTantra considers a constant flow of pumping for the design period of 30 years. This is the main limitation of its use in the design of direct pumping and combined pumping and gravity networks. This free software is a window format of the earlier BRANCH programme which was working on the DOS system. However, GIS-based operations are not possible on this software, as with most of the other free software on distribution network modelling and design.

The JalTantra software can be used for optimising the diameters of transmission mains. For 24×7 water supply, equalisation of residual pressures at FSL of service tanks is most important. Without equalisation of pressures, there would be an inequitable distribution of water to the service tanks. Thus, operational zones on lower elevations would get more water with excess pressure and those on higher elevations will get less water with less pressure. After making equalisation of residual heads at the FSL of the storage tanks receive water just equal to their design requirement. Hence, without equalisation design of the transmission main is incomplete.

Although the JalTantra software works on the Windows operating system, it is non-spatial. Hence, the user has to give data on the lengths of pipes and the elevation of nodes manually. In case a designer wishes to use modelling and simulation through freeware or commercial software, the traditional iterative method of design using GIS can be adopted.

(b) Hydraulic Model: The design can be made using GIS-based hydraulic model. The model can be prepared using freeware or commercially established software. The brief procedure is as below:

MBR, R1 supplies water to the five demand nodes (ESR nodes). The steps involved are shown in the flow chart shown in (Figure 6.6), in which J-2, J-3, J-4, J-5, and J-7 are the demand nodes (shown in red colour) representing the service tanks, and J-6 is the intermittent junction on a ridge with no demand. In the hydraulic model, elevations to be given at junctions J-2, J-3, J-4, J-5, and J-7 are the FSLs of respective ESRs, whereas ground elevations are given to the junctions J-1 and J-6, which are intermediate nodes (not demand nodes).

Normally, assumed diameters, lengths (in case of non-GIS), pipe material, lowest supply level (LSL) of MBR and FSL, and ultimate stage demands are fed to the demand nodes as data. After assigning the data, the hydraulic model is run. Required iterations are carried out by way of changing assumed diameters suitably by using the above general principles.

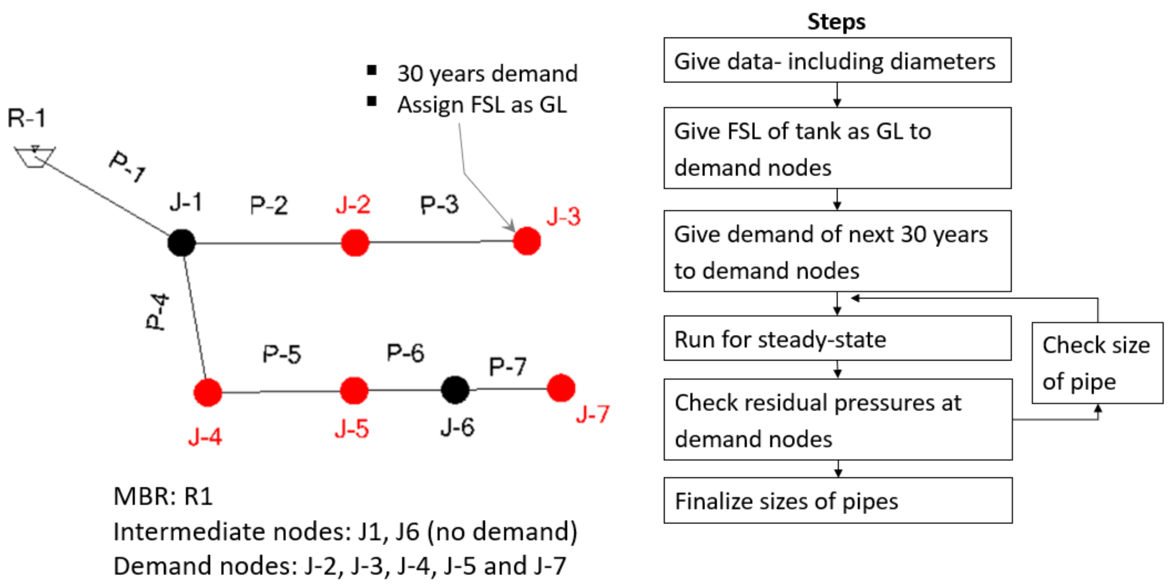


Figure 6.6: Iterative Design of Pipe Diameters of Gravity Transmission Mains

The software analyses the data and computes the residual head at the inlets (FSLs) of each ESR to be served by that MBR. The iterations are carried out till the residual head at FSL of some of the tanks becomes nearer to 3 m.

The iterative procedure for optimisation of diameters of any transmission main in a hydraulic model is shown in Figure 6.7 and is explained below:

1) After running the hydraulic model of the transmission main, we get two tables: (i) pipe table and (ii) junction (node) table. The pipe table contains pipe diameter, velocity, head loss (h_f), and head loss gradient (h_f/km). The junction (node) table contains residual nodal heads. During each iteration of the run of the hydraulic model, both the pipe table and junction tables are kept open so that the pipe diameters, its head loss (h_f) and head loss gradient (m/km) and the residual nodal pressures (m) can be observed simultaneously.

In the pipe table, sort diameters in descending order, and observe values of velocity and head loss, h_f (m/km) in adjoining columns of the junction table.

2) Decrease diameters of the pipes in which velocities are too low and whose diameter is more than 100mm and again run model.

3) Observe the values of velocities in the pipe table. If velocity is less than 1 m/s and h_f (m/km) is also less than 10 m/km and minimum nodal pressure is also more than or equal to residual nodal head as per norm (3 m), the steps are repeated.

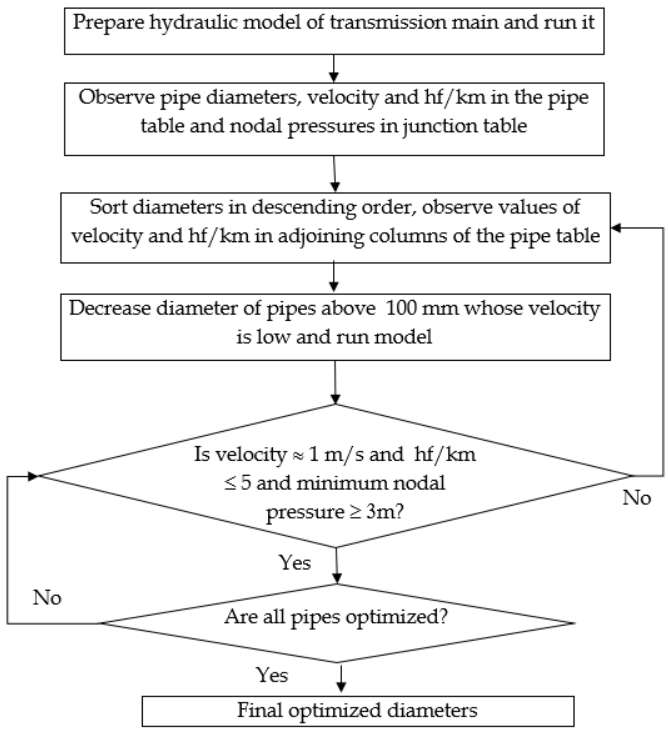


Figure 6.7: Iterative Process

4) The process is repeated for all the pipes whose diameters are more than 100 mm (which is minimum diameter), till we get all optimised diameters.

6.9 Complete Gravity Water Transmission Mains

In a complete gravity network, the supply of water is from MBR to various service reservoirs by gravity as shown in Figure 6.8. The MBR may be located at the ground level as shown in Figure 6.8 or may be elevated as in the case of a multi-level system involving ZBRs. These LSLs of ZBRs are determined considering topography and HGL requirements of reservoirs under respective ZBRs. Thus, the supply level at the source is arrived, which can be considered as the LSL in MBR.

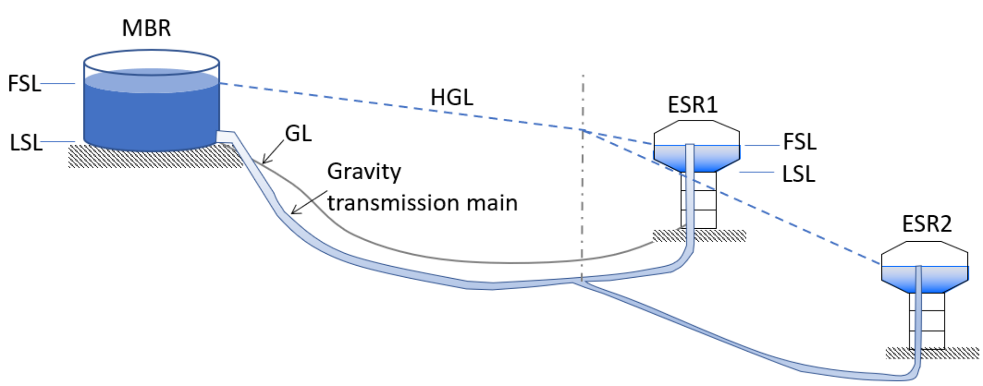


Figure 6.8: A Typical Gravity Transmission Main

6.9.1 General Principles of Design of Gravity Transmission Mains

In general, the following principles are to be adopted in the design of transmission mains by gravity:

(i) After designing optimised boundaries of operational zones of the distribution system, the LSLs of all tanks are known. By adding the necessary side water depth (SWD), we get FSLs of all ESRs. The transmission main shall be designed to give a minimum residual head of 3 m at FSLs of every service tank which is to be fed by the transmission main. The residual head should be as close as possible to 3 m so that quantity of water supplied to the service tank is nearly equal to the demand of the operational zone that the service tank is serving.

(ii) Grouping high-level and low-level service tanks (ESRs) in the city should be done, preferably by use of the GIS tool of the IDW surface. A case study of the grouping of low and high-level

ESRs is provided in **Annexure 6.2**.

(iii) Lower level group of ESRs should be fed from a low-level MBR and a higher level group of ESRs should be fed from a high-level MBR, through separate transmission networks so that only the needed quantity is pumped to the high-level MBR. This arrangement makes substantial saving in monthly recurring energy bills on account of pumping.

(iv) Several methods for the optimal design of branched networks are available. Linear Programming (LP) based models are most useful for the design of branched network as they provide global optimal solution considering discrete pipe sizes. In the case of Water Transmission Networks (WTN)s, link lengths are more and the residual head on each service reservoir is required to be equalised. The LP in this case not only minimises the cost but also tries to equalise the residual heads at the service reservoirs. Therefore, BRANCH and JALTANTRA software based on LP should be preferred for the optimal design of the transmission main network.

(v) The use of modelling and simulation by free software or commercial software, the traditional iterative method of design can be adopted, which is discussed in Figure 6.7

(vi) Any method based on a single size for each link will produce a higher residual head at each reservoir. Therefore, for WTN where residual head equalisation is a must, the sizes of the branch mains, if possible, should be partly reduced using the moving node method as described in the subsection 6.9.2.

(vii) Criteria for velocity (m/s) and head loss (hf in m/km) are discussed in Section 6.6.

(viii) The diameter of the transmission main on downstream of MBR should not be excessively more and can be little more than that of the inlet diameter of the pumping main (on the upstream side of MBR) feeding the MBR. As the pumping main has well designed economical diameter, it is used as a guiding factor.

(ix) If assumed diameters after analysis indicate that many ESRs get negative residual head, the MBR level needs to be suitably increased in case of new scheme. Thus, LSL of MBR and diameters of transmission main are arrived. Then following review should be taken:

(x) For minimising energy cost, it is necessary to lower down LSL of MBR to the extent possible, but this increases the capital cost due to increase in diameters in transmission main. For striking the balance following are the guidelines:

(xi) From the main network, there is an exclusive branch to feed the ESR at its end, and increasing or decreasing the diameter of that branch does not involve tangible capital expenditure, hence, the diameter of that branch can be increased or decreased to make the network hydraulically and energy cost wise efficient.

(xii) If in a large network of transmission main, if only one or two ESRs, that are yet to be constructed, show negative/insufficient residual head, then for such critical ESRs, the following arrangement may be considered:

- Decrease side water depth
- Increase the diameter of branch pipeline to the critical ESR, remember diameter of main lines should not be increased.
- Critically examine the LSL provided for that ESR and decrease it by a meter or so by attempting reduction in head loss in distribution of the relevant OZ/ DMA and by increasing diameter of the feeder main to DMA. Decrease of 1m in LSL of critical ESR leads to decrease of LSL of MBR by 1m, which makes sizable reduction in energy cost as total water needs to be lifted by decreased head of 1m.

- Lower the FSL of these ESRs suitably so that the designed quantity of water from MBR is assured to reach these ESRs and deficiency of nodal heads in the distribution system is redressed as under.
- provide an online pump on the outlet of such ESRs which will feed its service area; or
- provide a pump on the pipeline leading to the inlet of pressure deficient DMA served by that ESR.

This arrangement is more economical than increasing the level of MBR and pumping total water to high elevation or increasing the diameter in long lengths of the network. If the height of any of those ESRs needs to be decreased too much, then it is better to go for a sump and pump house. From the sump, water could be pumped to the ESR of that operational zone or directly pumped into the distribution system of that OZ.

(xiii) If the main line of the transmission network goes down the slope and again rises, then the ESRs on branches in the lower level will have a high and unrequired residual head. This scenario in the branch line feeding group of ESRs at a lower level is an indicator that the pumping energy is being wasted. On the other hand, while proposing alignment of the main line along a road on a high contour, care should be taken that the top of the pipeline is below HGL by 1 m at least at a critical place. This type of critical place also gives a signal for providing a sump and pumping to downstream ESRs on high locations. It also indicates for providing a ZBR and pumping water to it, for gravitating water to high-level ESRs. To ascertain this aspect, it is necessary to add nodes showing the elevation of ridge points.

6.9.2 Equalisation of Residual Head

In an ideal design of water transmission networks (WTNs), residual heads at all the ESRs should be the same as the minimum required ones. JalTantra has the capacity to produce such designs. However, because of the topological conditions, minimum pipe diameter conditions, and other inherent conditions in design, residual pressure at all the ESRs may not be observed the same. The performance of the system, when left to itself, would be different from the design one. In practice, the heads more than the minimum required ones increase the pipe discharges until the excess head becomes practically nil. In short, the performance of the system, in actuality, is head-dependent, rather than flow-dependent, as assumed in the design. In order to match the flow-dependent and head-dependent performances of the leading main system, it will be necessary to make the available flow rates equal to the required flow rates at different reservoirs. This can be achieved by dissipating the excess head in the leading mains supplying water to city/village service reservoirs by achieving equal residual head at FSLs of all ESRs. This will make the available flow rates practically the same as the desired ones at all service reservoirs.

The dissipation of the excess head and thereby flow adjustment can be achieved through the provision of pressure-reducing valves. Since they are costly and their fine-tuning to the desired level is difficult, some simple head-dissipating devices can be used. These are: (1) replacement of a part of the branch leading main by a smaller diameter pipe; (2) provision of one or more orifice plates; (3) partial closure of a valve in the branch leading main; or (4) a combination of the three. Since the head dissipation through partial replacement of the existing pipe by a smaller diameter pipe as well as that through the provision of orifice plates alone becomes a permanent solution and does not provide flexibility for easy adjustment in the future, they alone should not be used. For flexibility and fine tuning partial closure of inlet valve is also needed. The solution, therefore, should consist of one of the following measures:

(i) *Partial closure of valve*. When the head to be dissipated is small, a valve provided in the pipe can be partially closed so that the flow can be restricted to match the design flow. Herein, the valve is working as a head-dissipating device. Its adjustment, as recommended by the designer and to be fine-tuned during the trial run, will not be tampered with in the day-to-day operation of the system. Any adjustment that may be necessary in future for changed demands will be made by the central agency.

(ii) *Partial replacement of a branch leading main:* with a smaller diameter pipe is the best solution. Furthermore, it results in decrease in cost of branch leading main. When the head to be dissipated is extremely large and the discharge in the leading main is small, the number of orifice plates in solution 2 is excessive. For such situations, a solution consisting of (a) partial closure of a small diameter valve, (b) one or two orifice plates, and (c) partial replacement of the leading main by a small diameter pipe should be provided. Measure (a) would provide adjustment for fine-tuning during calibration, while measure (b) would help in the adjustment of discharge in the future if measure (a) alone is not sufficient.

(iii) *A combination of partial closure of a small diameter valve and orifice plates*. When the head to be dissipated is large, and the length of branch leading main is small, provision of only partial closure of a valve would not be advisable to dissipate the excess head. Herein, some orifice plates are used in addition to the partially closed valve.

Apart from the above three methods, the Moving Node method (if using a hydraulic model) is most effective.

6.9.3 Moving Node Method

Hydraulic models as well as evolutionary-based design techniques provide designs with a single size for each link and result in higher residual heads. The concept of a single pipe size for each link is understandable for water distribution networks, wherein nodes are closely located. In transmission mains, the distance between the nodes may be several kilometres. Therefore, to save on cost and reduce excess pressure, additional nodes can be generated, and part of the link can be replaced by smaller diameter pipes.

A simple method called as “moving node method” is proposed to achieve these dual objectives of reducing the cost and to equalise the heads. The method works iteratively and stops when residual heads at all the reservoirs are equalised.

From the main network of transmission main, every ESR/GSR has an exclusive branch that serves as an inlet to that ESR. The velocity (m/s) and hf (m/km) in this branch are to be increased by decreasing diameters for dissipating excess residual head. For this purpose, the length of the branch main should be divided (Figure 6.9) into two segments, say L1 and L2 by providing an extra node at the meeting point of (junction) of L1 and L2.

By assigning decreased diameters to the segment connecting the reservoir and by adjusting its length by moving the node at the junction of L1 and L2, the residual head is brought down as close as possible to 3 m. This needs to be repeated for each branch. An increase in velocity up to 4.3 m/s in a small length does not cause any problem as some extra margin is available above the criteria of a minimum 3 m residual head. The design obtained using the moving node method will have two sizes for each branch in the network. The logic of this process in the hydraulic model is shown in Figure 6.10.

The solution may not be exactly the same as obtained by LP-based algorithm but will be close to that and depends on the experience of the designer.

It may be noted that the suggested solution would require a minor adjustment in the field. This fine-tuning can be done during the trial runs. The head-dissipating devices (valves, orifice plates, and smaller diameter pipes, if any should preferably be located on branch lines near the downstream end of the transmission main. This will ensure the hydraulic gradient is above the centreline throughout, thus avoiding the formation of sub-atmospheric pressures in the leading mains.

However, when the head to be dissipated in a long leading main is large, orifice plates and reduced pipe lengths may be provided partly at intermediate places to avoid subjecting the entire leading main to large heads. The head-dissipating valve, however, should be provided at the downstream end. Spacing of at least 100-times the diameter of the leading main between adjacent head-dissipating devices should be used so that normal flow is established between adjacent head-dissipating devices.

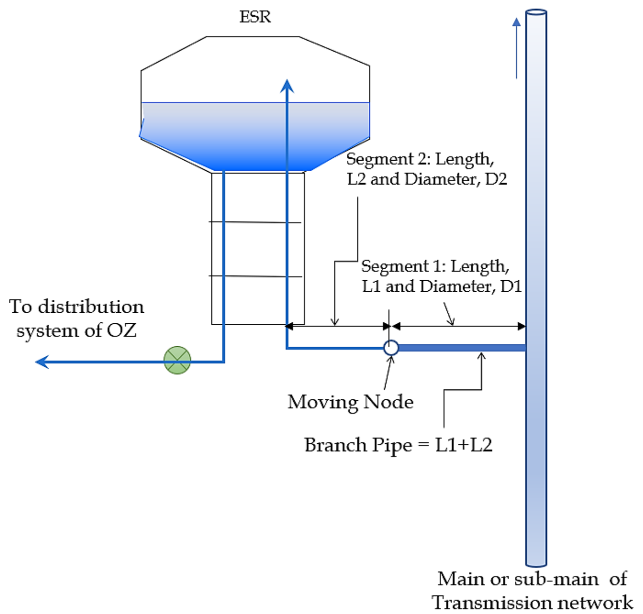


Figure 6.9: Branch Pipe with Two Segments

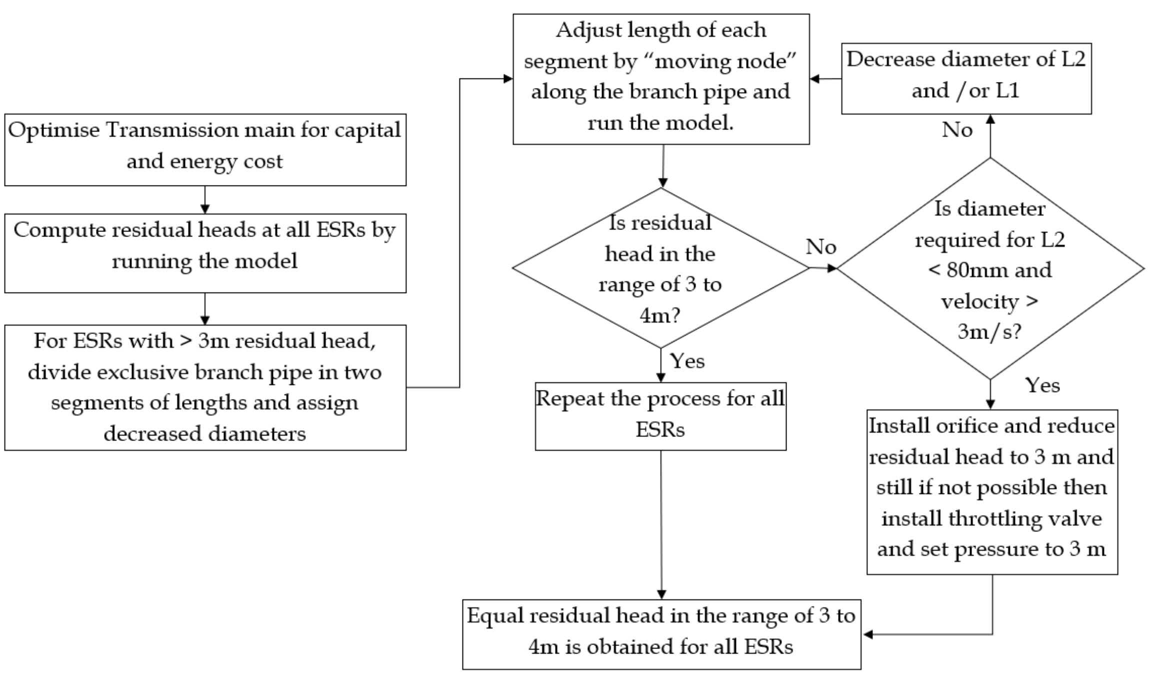


Figure 6.10: Logic of Making Equalisation of Residual Pressure

Designer on drawing should show a table showing hf/km and velocity in m/s for main stretches of transmission main so that the passing authority can visualise optimisation of cost.

A case study of complete gravity transmission main is presented in **Annexure 6.3**.

6.9.4 Manifold

Sometimes, it is desired to provide head-dissipating device on large diameter main pipelines, especially when larger than the required size is selected to restrict velocity/head loss gradient. This usually happens in hilly regions. Pipes are laid at high slopes and have excessive pressures. The dissipation of head can reduce excess pressure and controls the flow. However, the provision of a large diameter valve increases the cost of the network, and its operation in the field would be difficult. In such cases, a provision of pipe-valve assembly is more useful. Few such pipe-valve assemblies are installed in a Rural Regional Water Supply Scheme (RRWSS) supplying water to 2 towns, 'Daryapur' and 'Anjangaon', and 156 villages in Amravati District. The source of water for this RRWSS is Shahnoor Dam.

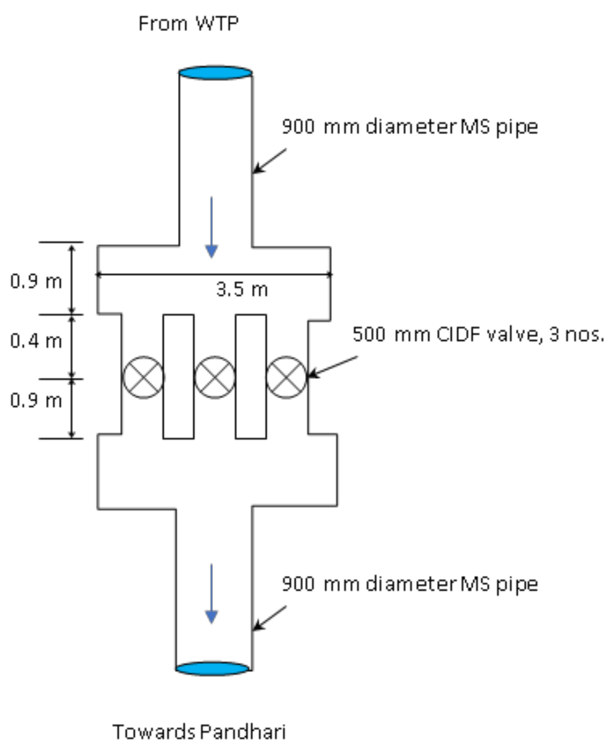


Figure 6.11: Pipe-Valve Assembly near Phandari Phata

Intake works are located on the canal from Shahnoor Dam. The scheme consists of the supply of water from the sump at WTP to 11 MBRs which in turn supplies to 103 villages ESRs. The flow from the intake to village reservoirs is completely by gravity. A typical pipe-valve assembly provided near 'Phandari Phata' on a 900 mm diameter pipe is shown in Figure 6.11. The assembly consists of three parallel pipes of 500 mm diameter pipes connected between two barrels of 900 mm size. One valve of 500 mm diameter is provided in each of the three 500 mm pipes. Instead of three parallel pipes, two pipes can also be used.

When the flow through large diameter more than 1000mm diameter pipeline needs to be controlled then this type of arrangement is important by incorporating a proper flow controlling mechanism apart from the isolation valve.

6.10 Design of Branched Pumping Mains

The branched pumping mains are of two types - direct pumping and combined pumping and gravity system.

6.10.1 Direct Pumping

It may not be possible to feed all ESRs by gravity from MBR/clear water sump at WTP. In that case, it is necessary to locate the sump at the appropriate place and pump water to the needed ESRs (Figure 6.12).

First preference should be given to pump water by separate pumps to separate ESRs by separate pumping main if ESRs are in different directions from the sump. In this case diameters of the pumping main work out to be less. If this arrangement is not possible, then a branched pumping main as shown in Figure 6.12 is the option.

If the pumping head is not much, it is desirable that combined pumping and gravity mains are used. In a combined system, water will be pumped to an MBR which in turn will supply to ESRs by gravity.

The methodology suggested for the economical design of pumping main to single MBR can be extended for the design of a direct pumped transmission main system feeding to multiple reservoirs (Figure 6.13), or a combined pumping and gravity transmission system. In a direct pumping system, an increase in the pumping head results in a decrease in network cost but increases the cost of the pump, and associated energy cost over its design period. Similarly, in a combined pumping and gravity system, with the increase in the height of MBR, network cost decreases but the pumping cost increases. Therefore, to arrive at the economical diameters in both cases, pipe cost, energy charges, and cost of pumps and other costs, as discussed above, should be considered. Two approaches: (i) using JalTantra software; and (ii) using the hydraulic model are discussed.

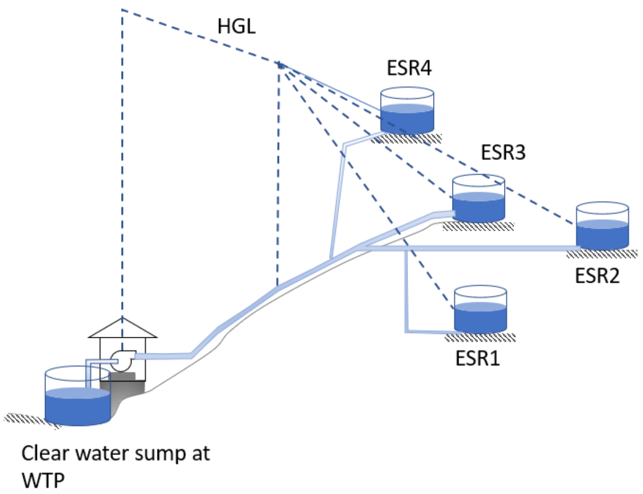


Figure 6.12: Branched Pumping Main

i. **Using JalTantra Software:** In case of a water transmission network (WTN), a single pumping main is now replaced by a network. As several pipes are to be sized and several options are available for each pipe, many combinations can be formed giving different pumping heads and different network costs. Evaluating all these alternatives to select the best alternative is difficult for most practical problems. Therefore, a combination of the Linear Programming based optimisation methodology for the design of branching networks with different source heads and the present worth (PW) method of economic analysis for comparing alternatives is recommended for the optimal design of WTN.

   The entire methodology consists of the following steps.

   a. Consider two stages of 15 years each and calculate the design flow at the end of each stage. Also, find the average flows for both stages.
   b. Select an initial trial value of the source hydraulic gradient level (HGL). This may be obtained by considering an average head loss (say 1.5 to 2 m/km) on the critical path and the minimum required HGL at the critical node including the residual head. (Critical path from source to any demand node is the path having the least available hydraulic slope, and the critical node is the node at the end of the critical path).
   c. Design a WTN for the selected source head using LP for the ultimate stage flow. The JalTantra software or any other LP or ILP-based model can also be used. Obtain the cost of the network. Check pipelines for the velocity and water hammer pressure criteria. Modify sizes or class of pipe, if necessary, and find the revised cost.
   d. Calculate the necessary pumping head for the selected value of the source head and obtain the pump capacity for the ultimate stage and its cost.
   e. Carry out analysis of the network for intermediate stage flows and obtain the necessary pumping head for the intermediate stage. Also, calculate the pump capacity and pump cost. Find PW of the pump cost.
   f. Obtain the number of hours of pump operation for the mean flow during both the stages considering the operation of pumps for the required number of hours at the end of the stage.

Calculate average annual energy charges and obtain PW of energy charges at the beginning of their stage.

g. Find the PW of the pipe cost, pump cost, and energy cost. PW of other cost components like operation and maintenance costs can also be obtained in a similar way.
h. Repeat steps (3) to (7) by adding a fixed increment to the source head. If the PW is found to be less than that of the previous alternative, continue further. Else, check by lowering the HGL value of the source head.

Initially, a higher increment can be taken to find an approximate value. Increment can be decreased for obtaining a more correct value.

ii. **Using hydraulic model:** A GIS-based hydraulic model can be effectively used for equalisation of residual pressures at FSL of service tanks. For carrying out the optimisation as well as equalisation, the hydraulic model needs to be prepared which can be prepared using any network freeware software or any commercial software. The advantage of using such software is that the transmission main can be mapped on GIS.

Equalising residual head at FSLs of ESRs is then achieved by a simple method called “moving node method”. By dissipating extra residual head and by bringing residual head to 3 to 4 m for all ESRs/GSRs, the storage tanks receive water just equal to their design requirement.

By equalisation of pressures at FSL of service tanks, a proper timetable of closing inlet valves can be enforced without allowing any stretch of transmission main from getting empty.

Two case studies of direct pumping are presented.

a. Non-spatial rural water supply scheme (RWSS) for multi-villages with optimisation of pipe cost and equalisation pressures at service reservoirs using JalTantra software. A case study of RWSS in Nadia District of West Bengal is presented in **Annexure 6.4**.
b. A GIS-based hydraulic model with optimisation of pipe cost and equalisation pressures at service reservoirs using established software. A case study of the Shirpur water supply scheme in the Dhule district of Maharashtra is presented in **Annexure 6.5**.

6.10.2 Combined Pumping and Gravity System

A combined pumping and gravity system is shown in Figure 6.13. In this system, water from the clear water tank (CWT) is pumped to the MBR, which then supplies water to various service reservoirs by gravity. The objective is to compute the optimum LSL of MBR for which the capitalised value of pipes and energy is the least.

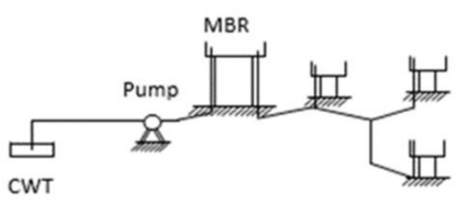


Figure 6.13: Combined Pumping & Gravity System

Optimisation of the cost of pipes and energy is done by using JalTantra software. A case study of one city representing a combined gravity and pumping system is presented in **Annexure 6.6**.

6.11 Interlinking of Transmission Mains from various sources for disaster management

Heavy rainfall causing floods and wash away of intake wells, lack of monsoon causing dried up source, silting of WTP through sand are some of the examples of disastrous condition in which no water is available from an affected source for some period of time. In such a situation, it is desired that water is made available to consumer from other nearby alternative sources. This requires linking of transmission mains from various sources.

6.11.1 Concept of Ring Main in Chennai

The City of Chennai, experiencing frequent draught, had implemented ring main system around its core area. Ring main system receives water from all the sources with an objective to maintain adequate water supply in different parts of the core area of the city in the event of failure of any surface water resource.

A schematic of the water supply from British era core city of 67 sq. km and its expansion initially to the expanded city and to the present Chennai Metropolitan Area (CMA) of 1189 sq. km, is shown in Fig. 6.14. The Red Hills Lake and its water treatment plant (WTP) gets inflows from Sholavaram Lake and this in turn gets inflows from the distant Poondi Lake (not in the drawing). The Chembarambakkam Lake and its WTP gets water from Veeranam Lake, 235 km down south (not in the drawing). The Poondi and Chembarambakkam Lakes are interconnected by a “level bedded canal” to “balance” the waters in these lakes in floods and droughts. The two seawater desalination plants (DSPs) are on the north and south ends. In the British era, it was only the Red Hills Lake gravitating the water to the city to a ground level reservoir (GLR) and pumped (by steam engine driven pump sets) to ESRs in the then three distribution zones. The later needs were to feed the new distribution zones in extended city and CMA and physically and functionally interconnect. This was with inputs from the World bank and other local funding institutions. The water from the WTPs, DSPs and other minor sources inject into the ring main along its alignment to keep it as hydraulically floating to facilitate drawls physically and functionally by valve controls to the various zones. The historical GLRs and pumping to ESRs are retained and all new zones are by “flat pumping” directly from GLRs (SUMPs) into their distribution system even from the 1990’s. This ring main system can be adapted in the old walled cities as also newer planning cities to command both inward and outward distribution from the ring main as a decentralised-centralised system.

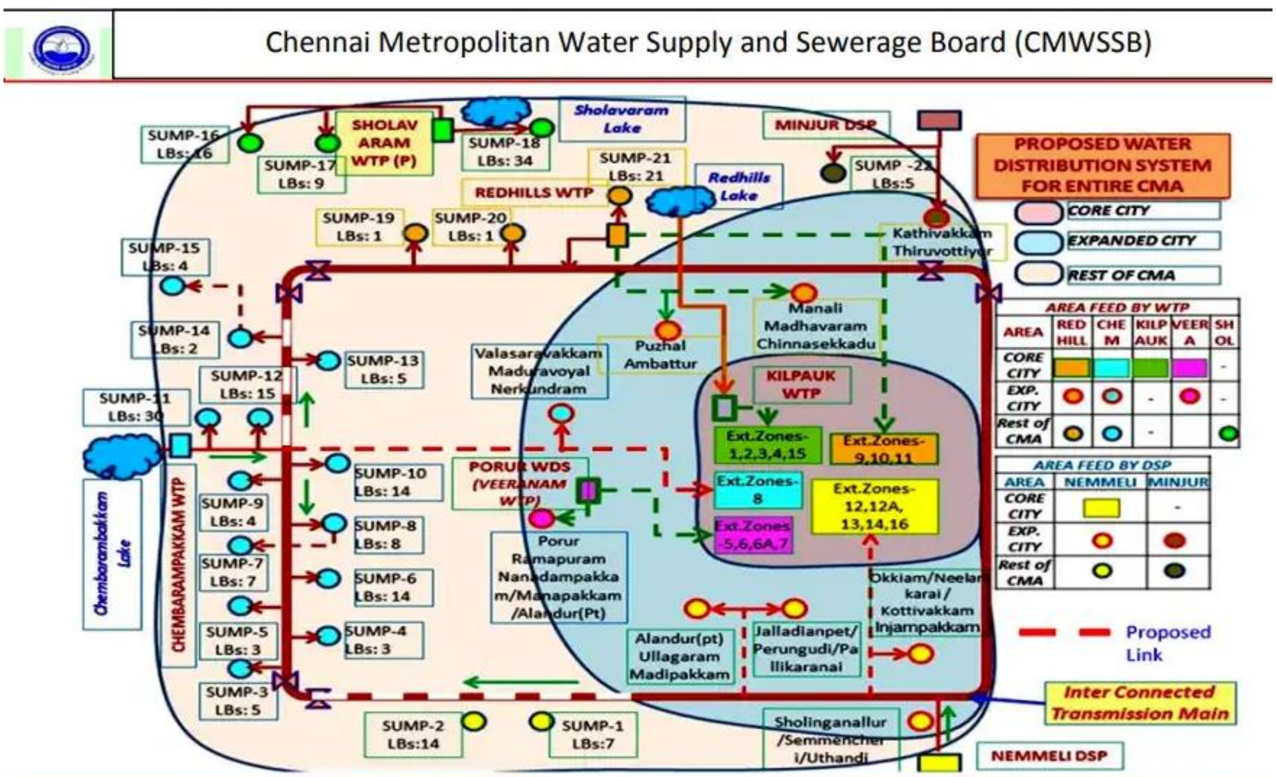


Figure 6.14: Chennai Ring Main Connecting Different Sources

(Source: Authors at Reference no. 7 of Bibliography for Chapter 6)

6.11.2 Interlinking of transmission mains in Mumbai Metropolitan Area

The City of Mumbai experienced a record-breaking 942 millimetres of rain in a period of 24 hours on 26 July 2005. The heavy monsoon rain triggered off deadly floods, which had disrupted the water supply scheme. The water supply of suburban towns was totally affected. Following this event, a disaster management plan was prepared and implemented in Mumbai metropolitan area covering 12 cities by interlinking transmission mains from various sources at various locations.

However, it is suggested that if the transmission system around any core area of the city or any other periphery area of the city fails, the concept of ring main may be adopted in a decentralised manner for different areas which are fed by at least two sources so that water will be available even if one source fails. Full supply from an alternate source cannot be guaranteed, however, the availability of 20% to 40% of supply from an alternate source can be planned.

6.12 Surge Protection for Pumped Transmission mains

Pumped transmission mains should be checked for water hammer analysis by any established software. A sample result of one such analysis is shown in Figure 6.15.

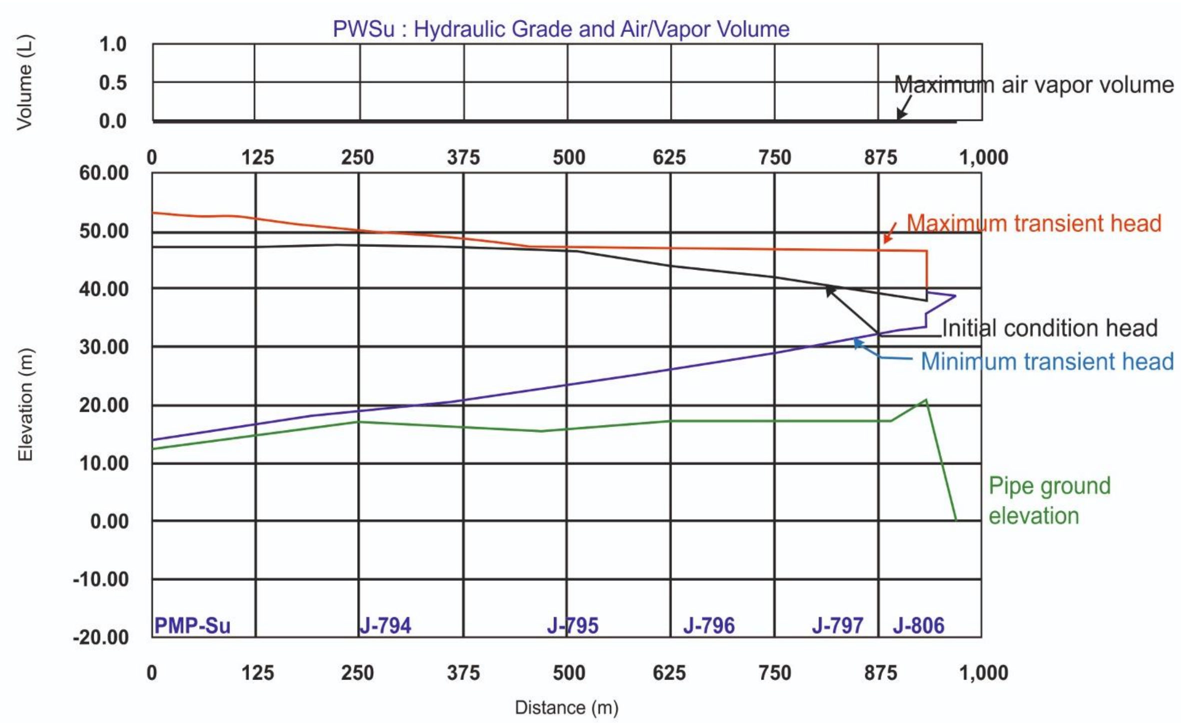


Figure 6.15: Sample Result of Water Hammer Analysis of a Pumping Main

It is to be observed that the minimum transient headline (shown in blue colour) must be above the ground elevation line (shown in green colour) of the pumping main which indicates that the pumping main is safe from cavitation. The pipe class should be such that it sustains maximum transient head shown in red colour. If the minimum transient headline (blue colour) happens to be below that of the ground elevation line (green colour) then the pipeline is unsafe. In such a situation water hammer protective equipment should be designed.

6.13 Minimisation of Energy Cost

Normally, the side water depth of MBR is 5 m, the inlet is at FSL, and the outlet is at LSL. However, it is recommended to keep invert levels of inlet and outlet at the same level, and the bottom of MBR with a non-return valve. LSL of MBR is lowered down to the extent possible. The bottom of MBR is placed further 1 m below the designed LSL of MBR as shown in Figure 6.16.

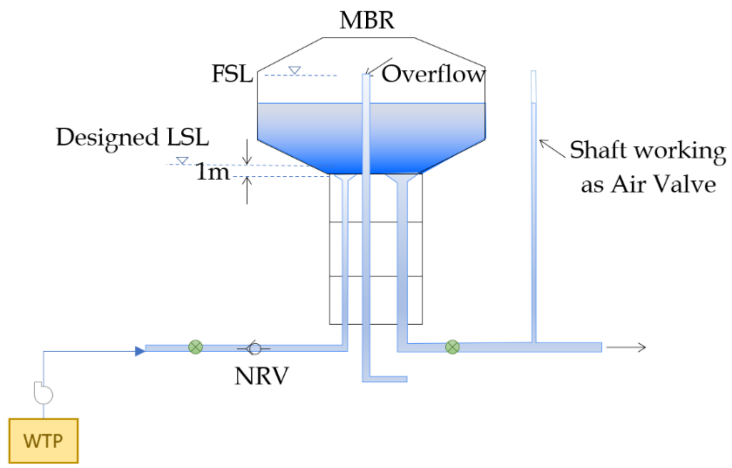


Figure 6.16: Inlet, Outlet Arrangement of MBR

This arrangement saves energy. We can save energy cost of pumping head perpetually, i.e., every month.

In case rising main to MBR leaks, then wastage of water due to emptying of MBR can be saved by shutting the pumps and closing the valve at the inlet.

In the case of MBR, if located on a hillock, i.e., on ground level, then the outlet of MBR should be with a bell mouth embedded below the bottom so that full capacity is available for use and MBR can be cleaned during maintenance. It is necessary to have all season road to MBR/BPT. Overflow pipe from FSL should discharge water at a place away from MBR and then that discharge should find its way to the natural stream.

6.14 Break Pressure Tank (BPT)

6.14.1 Merits of Introducing BPT

If a long pumping main encounters a hillock at a high altitude such that discharge on the downstream side of hillock/high-level ground can flow by gravity, then in such case advantage of topography can be taken by introducing a tank as BPT at such hillock. Even if high-level terrain is encountered such that HGL at the high-level ground is within 20-25 m above ground level, BPT can be introduced. The provision of BPT renders advantages as follows. Refer to Figure 6.17.

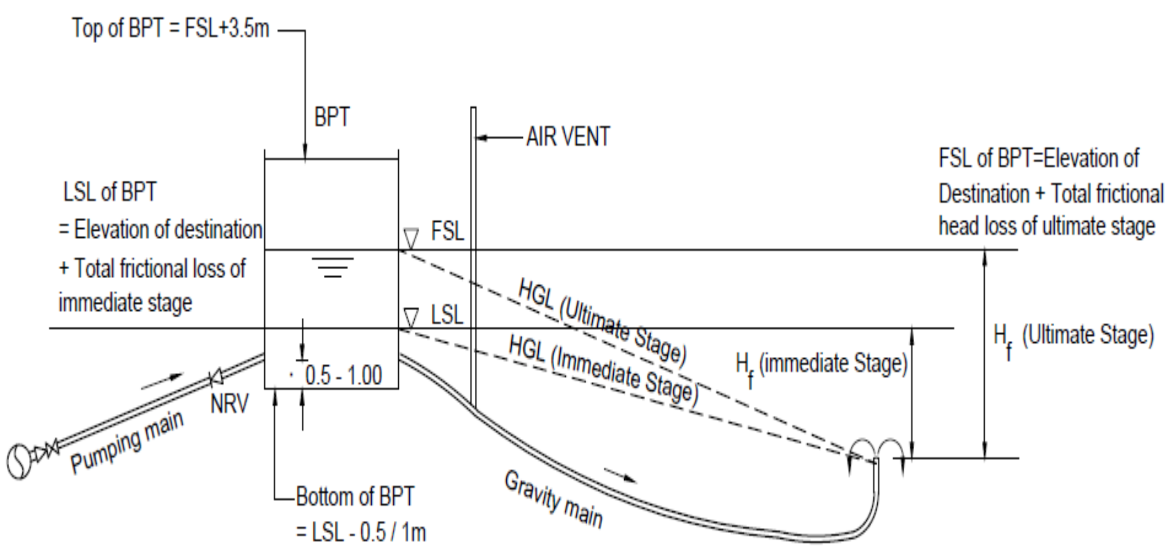


Figure 6.17: General Arrangement of Break Pressure Tank

i) No BPT Case

- In the absence of a BPT, the entire transmission from the pumping station to the destination would have functioned as a pumping main and would have to be designed for design pressure equal to the sum of operating pressure and water hammer pressure.
- The cost of such a high pressure pipeline shall be very high.
- The pipeline section at hillock becomes a critical stretch for sub-atmospheric pressures and consequent water column separation.

To overcome this critical aspect, well-designed and dependable water hammer protection device becomes essential to prevent the collapse of the pipeline due to sub-atmospheric pressure. The cost of such a water hammer protection device is usually high.

ii) On the introduction of BPT at hillock/high ground

- Due to BPT, the downstream pipeline functions as a gravity main. Thus, the downstream pipeline shall be totally free from water hammer pressures. A lower class of pipeline or lower thickness can be selected resulting in large savings in capital cost.
- The length of the pumping main is reduced from the pumping station to BPT. Cost of water hammer protection device for reduced length of pumping main shall also be less particularly as a critical section on hillock vulnerable to sub-atmospheric pressure and water column separation is no more applicable due to locating BPT at such section.

6.14.2 Improvisation by Manipulating BPT Location

It is not necessary that BPT location at intermittent hillock or high ground is a must. If suitable hillock or high-level ground is available at a short distance from the pumping station, such that HGL at such high ground is within 20-25 m above ground level, BPT can be introduced at such place. This arrangement converts the maximum length from the pumping main to the gravity main.

Figure 6.18 shows the theoretical location of BPT on enroute hillock at 15.5 km out of a total 56.5 km transmission main due to which 15.5 km becomes the pumping main and 40 km as the gravity main.

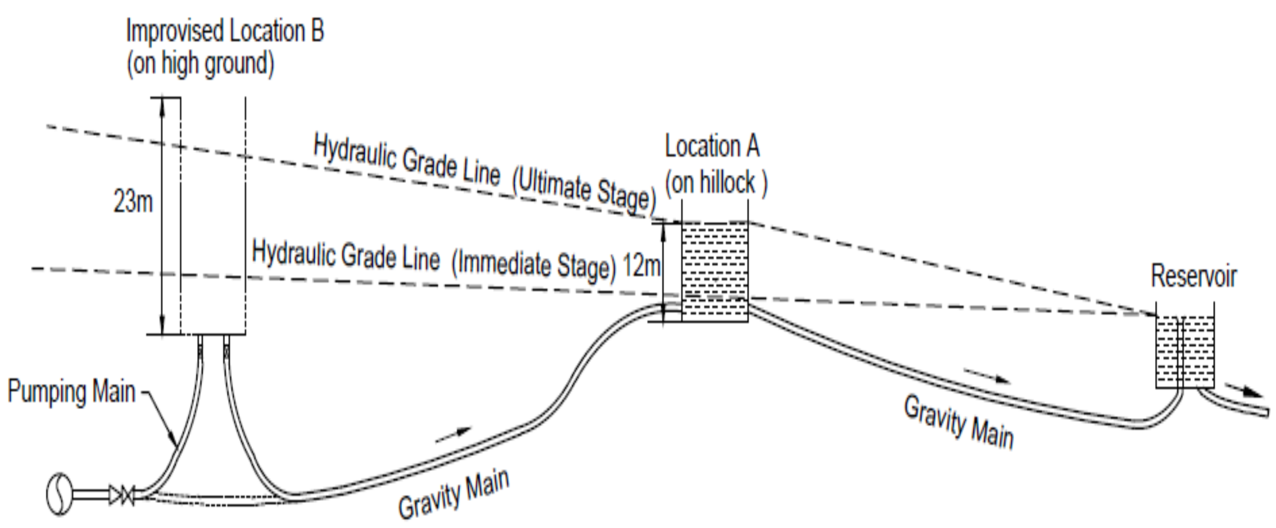


Figure 6.18: Improvised Location of BPT for Increasing length of Gravity Main

On improvisation by application of the principle, in the scheme for the city, a revised location of BPT is kept at the nearby high ground at a chainage of 1.5 km. A BPT of 8.6 m diameter × 23 m height is constructed (Figure 6.19) due to which length of pumping main is now reduced to 1.5 km and 55 km length functions as gravity main.

In another scheme with a pump head of 56 m and 600 MLD flow, the entire 40 km long transmission main is laid in plain terrain. The advantage of the availability of hillock is availed for locating BPT near the pumping station. Due to BPT, 39.8 km pipeline functions as gravity main and length of pumping main reduced to a mere 200 m. Thus, a significant savings in capital cost of pipeline, as well as water hammer control device, could be achieved.

Figure 6.19: 23 m high BPT nearing Completion

6.14.3 Usual Mistakes in BPT Design

i) Common Observation on Capacity: It is observed that in the absence of guidelines, often the size of BPT is arrived at from consideration of the volume required to store water at a steady design discharge for an arbitrary period. The arbitrary period is decided based on the experience of the designer, which could be a wild guess like 5 minutes, 10 minutes, or 15 minutes. Because of the fear that the size may become inadequate, BPTs much larger sizes than required are provided in various schemes.

The cross-sectional area of a BPT can be calculated, and if guidelines are followed, a BPT of much smaller capacity ranging from 2 minutes to 10 minutes can be adequate. Many BPTs designed, as per the guidelines, are functioning. A detailed discussion is as follows.

ii) LSL and FSL: Another usual mistake in designing BPT on a similar basis applicable for service reservoir, keeping LSL with friction losses for the ultimate stage. The result is that BPT admits and passes full flow at LSL only and the BPT runs practically dry. Hence, LSL is to be designed considering Hazen-Williams' 'C' for new pipes and FSL is to be designed for friction losses for Hazen-Williams' 'C' values for the old pipe.

iii) Incorrectly terminating inlet pipe at FSL: Similar to the service reservoir, the inlet pipe is terminated at FSL. In the initial stage, WL in BPT is at LSL. Due to the termination of the inlet pipe at FSL, the pump discharges at FSL whereas WL in the tank is at LSL, resulting in an unnecessary increase in the pump head equal to the design water depth of the tank. Hence, both inlet and outlet pipes shall be terminated at the LSL of BPT.

iv) Misunderstanding about Q_{in} and Q_{out} and balancing storage: In BPT, Q_{in} and Q_{out} are always the same irrespective of demand in the distribution system. In the case of ESR, Q_{in} is always constant, but Q_{out} varies from 20% to 250%-300%, depending on the lean hour and peak hour demand. Hence, balancing storage as per the mass diagram is provided in the service reservoir. However, balancing storage in BPT is not applicable.

v) Misconception about the increase in pump head due to BPT: Generally, the misconception is observed that due to the introduction of BPT, pump head increases. There is practically no change in HGL as well as pump head due to the introduction of BPT, as the inlet is kept at the level of the outlet. Only exit loss at the inlet and entrance loss at the outlet is added, the magnitude of which is very low - about 0.1-0.2 m, which is insignificant.

6.14.4 Hydraulic Design of BPT

Design objectives of BPT can be stated as follows:

i) BPT should never overflow during starting of pumps and normal steady state operation over the entire service period of BPT from the initial stage when the 'C' value is better, immediate stage, and ultimate stage when the 'C' value is the lowest due to deterioration.

ii) During starting of the pump, when standstill water in the downstream pipeline starts flowing, velocity is accelerated from V=0, causing WL to rise till steady state velocity, $V_{o,}$ is attained. During this acceleration period, WL attained may be higher than steady state WL. Even under this period, overflow should not occur.

iii) Under no circumstances should the head-on pump be wasted. This objective can be achieved by terminating the inlet and outlet at same level as discussed in the subsequent subsections.

iv) BPT should never be dry or fully empty. Generally, the tank is in RCC or steel construction. Concrete deteriorates if dry and steel tanks get corroded if subjected to dry and wet situations.

Design aspects

(i) Variations in design basis

The design of BPT depends on the profile of the downstream pipeline, the water content in the pipeline under standstill conditions achieved after stoppage of pumps (usually called no-flow condition), and flow characteristics during starting of pumps in multi-pump installation.

(ii) Categories of gravity main on the downstream side of BPT

The pipeline on the downstream side of BPT, i.e., gravity main, can be classified into three categories depending on the characteristics of the pipeline which include the longitudinal profile of the pipeline, average slope of the pipeline, and slope of hydraulic grade line (HGL).

a. Category-I: Refer Figure 6.20:

When the average slope of gravity main is greater than the slope of HGL, some length of pipeline from BPT will run partially full. BPT will remain empty all the time. Providing large size BPT, in this case, is not required and BPT with the nominal size is enough. In order to ensure that BPT is not dry, the outlet should be kept at least 0.5 m above the bottom of the tank.

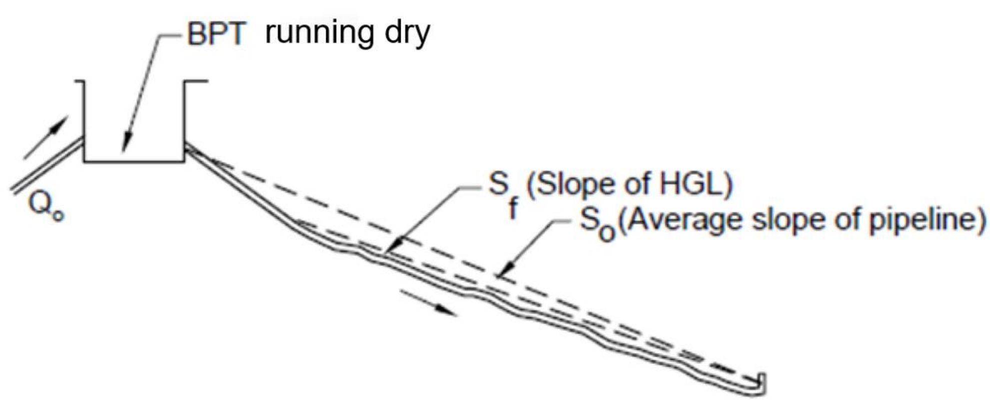


Figure 6.20: Category-I: S_0 (Average slope of pipeline > S_f (Slope of HGL)

b. Category-II: Refer Figure 6.21

In another case, the average slope of gravity main is less than the slope of HGL, and the longitudinal profile of the pipeline is such that during no-flow, the pipeline remains empty as the water is drained out due to a continuous downward slope after stopping inflow into BPT. In this case, when the inflow to BPT starts, water enters the pipeline and process of filling up of pipeline begins and the water level in the pipeline starts rising. Simultaneously, the velocity of water in the pipeline increases gradually. Thus, the water level will reach a steady state position gradually and will remain stationary at that position. In this case, a large size BPT is not required; nominal size is enough.

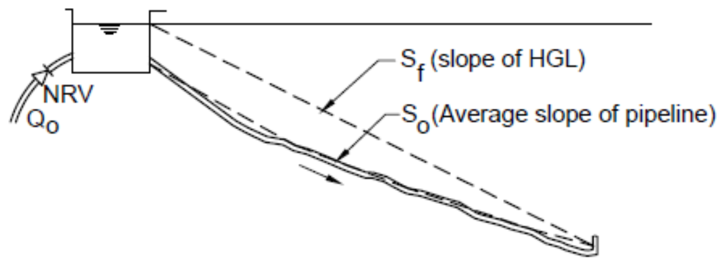
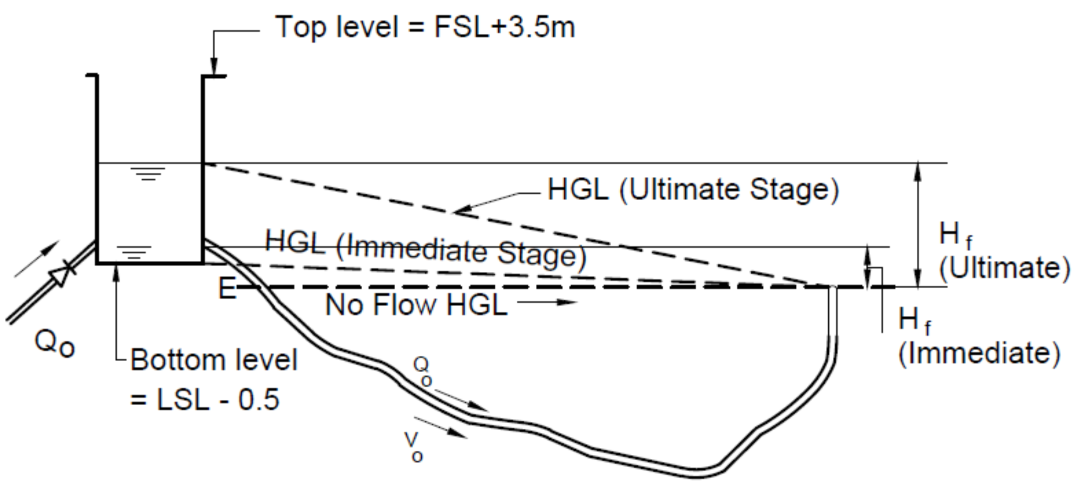


Figure 6.21: Category-II: S_o (Average slope of pipeline < S_f f (slope of HGL)

c. Category-III: Refer Figure 6.22

In the third case, the average slope of the pipeline is less than the slope of the hydraulic grade line (HGL), but the longitudinal profile is in the form of an inverted siphon. The pipeline remains practically filled with water after the stoppage of pumps.

This case is vital for detailed design and therefore, elaborated covering all pertinent design aspects.



L = Length of pipeline
Q_o = Steady discharge in the pipe line
V_o = Steady slate velocity in the pipe line

Figure 6.22: Category-III: Pipeline in form of inverted siphon

(Pipeline practically full under no-flow conditions)

(iii) Terminating inlet and outlet pipes in BPT

The outlet of the pumping main (which is inlet of BPT) and inlet of the gravity main (which is outlet of BPT) shall be kept at the same level and should be marginally above the bottom of BPT as shown in Figure 6.21.

This will save additional head on the pumps which otherwise would have come if the outlet of the pumping main is kept above FSL of BPT. By this arrangement, the advantage is that the water level in the BPT can rise to such a level that the driving head is just sufficient to negotiate the frictional losses occurring in the gravity main for the immediate stage and also the ultimate stage. This will save energy costs for both the immediate stage and ultimate stage. The water level will increase just to the required level and the energy cost can be saved.

Usually, the top of the inlet is kept at FSL in the reservoir on the reasoning that if a burst or major leakage occurs in the pumping main, water in the reservoir should not drain causing water logging at the leakage location. This reasoning can be acceptable for service reservoirs or MBRs where capacities are four to eight hours. In BPT, however, capacity is much less, i.e., few minutes.

However, even to prevent such draining of BPT, a non-return valve (NRV) can be provided on the incoming pipeline to allow inflow to tank; but prevent reverse flow as shown in Figure 6.20. Head loss in NRV is 0.15-0.3 m which is insignificant compared to the average saving in pump head by about 3-5 m.

(iv) Deciding the Lowest Supply Level (LSL) of BPT

Initially, the driving head required to pass the intermediate stage (base year+15) flow through the gravity main is required to be computed which is equal to the elevation of the destination, in this case, the elevation of the lip of the aerator plus the total frictional head (including minor losses) of intermediate flow as shown in Figure 6.21. Care should be taken that while working out the frictional head loss, the C-value of the new pipe (highest C-value) should be taken. Thus, the LSL of BPT is decided. The bottom of BPT should be minimum of 0.5 m below LSL to ensure that the tank never remains dry.

(v) Deciding FSL and Height of BPT

Initially, the frictional head loss (including minor losses) for the ultimate flow that the gravity main can pass should be computed. Care should be taken that while working out the frictional head loss, the C-value of the old pipe (lowest C-value) should be taken. FSL of BPT is then elevation of destination plus the frictional head due to ultimate flow (including minor losses). Considering the safety of 2 m against overflowing, the height of BPT is then computed as,

Top of BPT = FSL + 2.5 m (including free board of 0.5) (6.8)

(vi) Area of cross-section of BPT

V.N.I.T., Nagpur has developed guidelines for sizing BPT, based on the equation of continuity and equation of motion, the equation for the cross-sectional area of BPT is developed which is given by:

$$A_T = \frac{4AL}{F^2 V_o^2 g} \quad (6.9)$$

Where: A_T = Cross-section area of BPT; A = Cross-section area of downstream gravity pipe; D = diameter of gravity pipe; F = fL/(2gD) = friction loss constant; g = gravitational acceleration; L = length of pipeline; V_0 = steady state velocity in the pipeline.

$$h_f = FV_o^2$$

Or,

$$F = \frac{h_f}{V_o^2} \quad (6.10)$$

Here h_f can be computed using the Hazen-Williams Formula.

Optimisation of BPT can be done by reducing A_T (cross-section area of BPT) by 20%-30% in which case, small WL rise above steady state WL may occur. However, this small rise can be accommodated in a safety margin kept above FSL.

In essence, the following should be adopted for inlet and outlet pipes for all above three cases:

a) Inlet and outlet should be kept at the same elevation.

b) LSL of BPT should be computed for present stage demand with C-value of new pipes and FSL is computed with ultimate stage demand and C-value of the old pipe.

c) Every design including hydraulic modelling always has a factor of safety. In the design of the pipeline, the factor of safety is in terms of a slightly higher designed LSL of MBR. Design LSL is of course not to be lowered down but unnecessary pumping costs can be saved. This can be done by providing the bottom of the slab at an elevation lower by 1 to 2 m below the design LSL. In the steady state of operation, i.e., inflow equal to outflow, the water level will not climb up to the designed LSL but will remain at a level lower than that and the pumps will operate for this decreased head. This yields in saving on electricity bills due to a decrease in the head of the pump by more than 5 m. The decrease in the head due to this arrangement compared to the inlet at FSL is 5 to 7 m. This is an extra saving over and above saving. If the head of the pump on the inlet pipe is 50 m, then the saving is about 10% to 14%. For lengths of transmission mains up to 10 km, provide the bottom of MBR at 1 m below the design LSL and for more lengths bottom of MBR should be 2 m below the designed LSL.

A typical design of BPT is illustrated in **Annexure 6.7** using the data of the water supply scheme of one city.

6.15 Thrust Block

It is necessary to provide thrust block (Figure 6.23) in the shape of concrete blocks to resist the forces that cause the pipe to pull apart at bends or other points of unbalanced pressure or when they are laid on steep gradients and resistance of their joints against longitudinal stresses is either exceeded or inadequate. Adequate anchor bars must be provided as per the site conditions embedded in concrete blocks to give additional strength and stability.

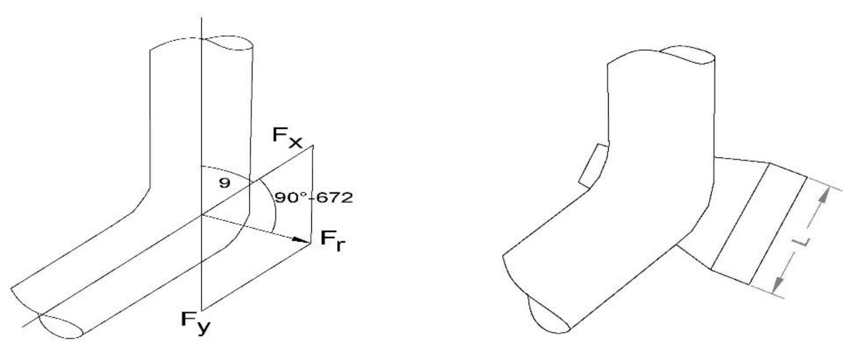


Figure 6.23: Thrust at a Bend & Thrust Block

Thrust blocks made of concrete generally in rectangular shape resist the unbalanced horizontal thrust to pull out the bend or pipe by counteracting the following forces.

(i) Weight of the block + weight of water in the enclosed pipe in the block
(ii) Friction resistance by soil
(iii) Lateral pressure acting on the block by soil mass
(iv) Lateral resistance of soil mass on the outer face of the projected pipe

Horizontal thrust caused by unbalanced static pressure by water at the bend,

$$F_p = 2PA\sin(\phi/2) \quad (6.11)$$

Where P = Internal water pressure in the pipeline

A = Area of cross-section of pipe

ϕ = Degree of bend angle

Counteracting forces to resist the horizontal thrust

It is as below:

(i) Weight of concrete block = Length × Breadth × Height × weight of concrete/unit volume
(ii) Weight of water in the pipe enclosed in Cement Concrete block = cross-section area of pipe × Length of pipe × wt. of water/unit volume
(iii) Weight of earth cushion over the concrete block= Width of block × height of the earth cushion × pipe diameter × weight of earth/unit volume

The lateral resistance offered by soil friction against the thrust block = (A + B + C) × Frictional resistance of soil

Lateral resistance of soil against the thrust block,

$$F_p = \gamma_s \frac{H^2}{2} L \left[\frac{1+\sin\theta}{1-\sin\theta}\right] + 2CHL\sqrt{\frac{1+\sin\theta}{1-\sin\theta}} \quad (6.12)$$

The maximum resisting pressure a soil mass will offer is termed the passive resistance and is given by:

$$f_p = \gamma_s h \left[\frac{1+\sin\theta}{1-\sin\theta}\right] + 2C\sqrt{\frac{1+\sin\theta}{1-\sin\theta}} \quad (6.13)$$

This maximum possible resistance will only be developed if the thrust block is able to move into the soil mass slightly. The corresponding maximum soil pressure is termed passive pressure. The minimum pressure which may occur on the thrust block is the active pressure, which may develop if the thrust block were free to yield away from the soil mass.

$$f_\alpha = \gamma_s h \left[\frac{1-\sin\theta}{1+\sin\theta}\right] - 2C\sqrt{\frac{1-\sin\theta}{1+\sin\theta}} \quad (6.14)$$

F_p, f_p = Lateral resistance of soil against the thrust block; γ_s= soil density; h = depth in m, θ = angle of friction in degrees, C = cohesion of soil (C = 0 for gravel and sand, 0.007 for silt, 0.035 for dense clay, and 0.15 for soft saturated clay), H = height of thrust block and L = the length of thrust block

Total counteracting forces by concrete block at bend should be ≥1.5. For the safe design of the thrust block, the factor of safety is 1.5. The minimum reinforcement in all thrust blocks should be provided 5 kg/m2. The spacing of these bars should not exceed 500 mm c/c.

In the case of end caps, either a thrust block at the end cap is required or the end cap should be dish-shaped like the ends of the air vessel.

A typical design of thrust block is given in **Annexure 6.8**.

Anchorages for Sloping Pipelines

Thrust block on slopping ground (Figure 6.24) is described by a step-by-step design guide (Thorley, 1994) for thrust blocks. It mentions restraining the forces generated by changes in direction of fluid flow in joint buried pressure pipeline networks.

Where buried pipes are laid in a straight line on slopes, a component of the dead weight of the full pipeline acts axially, increasing with the angle of the slope. This axial force pushes the pipes to slide down the slope. The design should prevent such movement from occurring.

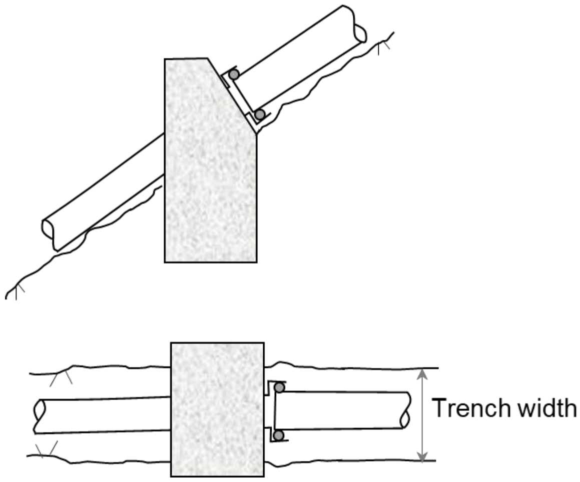


Figure 6.24: Typical Thrust Block on Sloping Ground

Pipes laid on shallow slopes do not slide due to the frictional resistance of the soil. However, if the pipeline is loosely wrapped with a polyethylene sleeve, the resistance becomes less and there is a chance to slide. So also, when slopes are such that it generates a sliding force more than frictional resistance, we need to support the pipeline with concrete anchors with integral keys or even by raking piles for slopes more than 1 in 4.

Concrete walls surrounding the pipe should extend at least half the pipe diameter above the crown and below the underside of the pipe and beyond the trench walls into the undisturbed ground on either side and be of suitable thickness to develop the required bond and to accept the shear and bending moments generated (Figure 6.24).

Pipes should be laid with their sockets facing uphill, and support structures located so that the external shoulder of the socket of each pipe bears against the pipe support. In this way puddles, flanges, or other securing devices are not required. Each pipe should be anchored. The use of anchored or self-restrained joints as an alternative should be considered irrespective of the pipe materials.

Proper attention should be given to preventing the erosion of the bedding material beneath the pipe. On long slopes, and depending on the gradient, more than one thrust block will be required. Table 6.8, taken from recommendations by Stanton Pipes for cast iron pipelines, gives spacing for the thrust blocks.

Table 6.8: Spacing for Thrust Blocks on Long Slopes

Gradient	Spacing for Thrust Blocks
1 in 2	5.5 m
1 in 3	11.0 m
1 in 4	11.0 m
1 in 5	16.5 m
1 in 6	22.0 m

Source: *("Guide for thrust blocks for buried pipelines," A.R. D. Thorley and J.H. Atkinson, published by CIRIA in conjunction with T. Telford, London, 1994)*

6.16 Surge Phenomenon and Selection of Surge Protection Devices

6.16.1 Occurrence of Surge and Causes

If discharge in a pipeline suddenly or rapidly changes, causing sudden or rapid changes in flow velocity, consequently a pressure wave occurs which propagates in the pipeline at acoustic speed both in forward and reverse directions. It lasts till the wave dies due to friction in the pipeline. Due to the propagation of pressure waves both the phenomenon, i.e., pressure drop due to down surge and pressure rise due to upsurge, occur in succession in the pipeline. The pressure drop and pressure rise are termed as surge pressures or water hammer pressures.

The change in discharge can be caused by the following operations or events.

i) sudden/abrupt opening or closing of the valve;
ii) power failure to pump motor sets either due to electric supply interruption or tripping of breaker/fuse failure on incoming switchgear;
iii) sudden stoppage of one pump in multi-pump installation due to any reason, may be tripping of power supply or motor stalling;
iv) starting or stopping of first and subsequent pumps in multi-pump installation;
v) sudden falling of gate of sluice valve installed in-line.
vi) due to the slamming of check valve.

Out of the above causes, the causes (i) and (iv) can be controlled and causes (v) and (vi) can be prevented by following suitable procedures as discussed in the sections below.

However, power failure and single pump sudden stoppage are beyond control. These two cases are very critical as discussed below.

6.16.2 Effects of Surge Pressure

The surge pressure wave travels and subjects piping system and other facilities to cycles of transient high and low pressure occurrences. These pressures and phenomenon can have several adverse effects on the piping system. If the transient pressure is extremely high, the pressure rating of the pipe may be exceeded causing failure through the pipe or joint rupture. Such a flow variation causing pressure can also lead to significant pressure reduction during wave travel in forward and reverse directions. If sub-atmospheric pressure condition results, the risk of pipeline collapse increases. Even if the pipeline does not collapse, column separation could occur if the pressure in the pipeline is reduced to the vapour pressure of the liquid. This causes the formation of vapour pockets which collapse when two separated water columns rejoin at high velocities. The collapse of the vapour pocket/cavity can in turn cause severe high pressure and rupture in the pipeline.

6.16.3 Preventing Surges in Starting and Stopping Operation of Pumps and Valves

The basic criterion in surge control is that rate of discharge change shall be such that the operation time of the valve is greater than Tc, i.e., 2 L/c. Here, L = Length of pipeline; c = Pressure wave propagation speed; Tc = Critical time for wave travel in forward and return directions.

Operating procedure as under shall be followed to prevent surges.

i) Delivery valve, whether sluice valve or butterfly valve, should be opened or closed slowly with a uniform speed of opening/closing so as to exceed the time of closing or opening above 2L/c.

ii) The operating speed of the valve actuator, electric or pneumatic, shall be slow to prevent rapid opening or closure and exceed time above 2L/c.

iii) Second and subsequent pump should be started or stopped in sequential order after allowing adequate time for previous pump operation to steady state condition and checking that the pressure gauge reading is steady (usually 10-second time interval per km length of pumping main is adequate).

iv) Overcurrent relay setting and/or fuse rating of incoming breaker and/or switch fuse unit shall be checked periodically.

Starting and stopping of pumps and opening and closing of the valve can be controlled and thus their ill effects can be avoided. However, power failure is beyond control and hence the suitability of pipeline and appurtenances need to be appropriately designed or protection devices provided.

v) Sudden falling of gate can be prevented by periodical checking of line valve/sectional valve.

6.16.4 Magnitude of Surge Pressure

The magnitude of surge pressure is additive/deductive to and from the normal pressure in the pipe and depends on the elastic properties of the liquid and the pipe and the magnitude and rapidity of change in velocity.

Maximum surge pressure (which occurs at the critical time of closure Tc or any time less than Tc) is given by the expression,

$$h_{srg} = cV_o/g \qquad (6.15)$$

Where, h_{srg} = maximum pressure rise or fall (upsurge and down surge) in m; c = speed of pressure wave propagation in m/s (also called celerity); g = acceleration due to gravity in m/s^2; V_o = normal velocity in the pipeline in m/s

Speed of pressure wave propagation is given by,

$$c = \frac{1425}{\sqrt{1+\frac{kd}{Et}}} \qquad (6.16)$$

Where, k = bulk modulus of water (2.07 × 10^8 kg/m^2), d = diameter of pipe in m, t = wall thickness of pipe in m, and E = modulus of elasticity of pipe material in kg/m^2 (Refer Table 6.9 below).

Table 6.9: Values of E for Different Materials

Material	E (Kg/m$^{2)}$
Polyethylene - soft	1.2×10^7
Polyethylene - hard	9×10^7
PVC	3×10^8
Cast iron	7.5×10^9
Ductile iron	1.7×10^{10}
Wrought iron	1.8×10^{10}
Steel	2.1×10^{10}
Asbestos cement	3×10^9
Concrete	2.8×10^9
Reinforced cement concrete	3.1×10^9
PSC	3.5×10^9

If the actual time of closure T is greater than the critical time Tc, the actual surge pressure is reduced approximately in proportion to Tc/T.

Surge pressure wave speed may be as high as 1,370 m/s for a rigid pipe or as moderate as 850-1,100 m/s for a steel pipe, and for polyethylene and PVC pipes, may be as low as 200-400 m/s.

6.16.5 Resultant Pressure on Occurrence of Surge Pressures

As stated in 6.16.1, the surge pressure can be down surge and/or upsurge. The surge pressure is subtractive from operating pressure as well as additive and occurs in succession.

Resultant pressure, H_{max} / H_{min} in the pipe system is thus:

During down surge; $H_{min} = H_o - h_{srg}$ (subject to vapor pressure limit)

During upsurge; $H_{max} = H_o + h_{srg}$

Where:

H_{min} = Resultant pressure during down surge;

H_o = Normal/Operating pressure;
H_{max} = Resultant pressure during upsurge

H_{min} however cannot fall below water vapor pressure level as the water vaporises. Refer to 6.16.6 (a) below for further discussion.

6.16.6 Surge Phenomenon due to Power Failure on Pumps

This is a most critical and key surge phenomenon and surge analysis, and selection of surge protection devices aim at protection from effects of down surge and upsurge for this vital event. When the power supply fails, the motor speed reduces rapidly. The rate of speed reduction depends on steady state torque and inertia of the pump motor set. A small pump motor set decelerates very rapidly whereas the rate of deceleration is slower in the case of a large pump motor set. Consequent to a reduction in motor speed, Q and H also reduce generally following affinity laws. Due to head drop, a down surge pressure wave travels along the pumping main towards discharging end at wave speed, c. At discharging end, forward flow velocity V_o becomes zero, and subsequently reverse flow occurs at velocity - V_o. Simultaneously, the wave gets reflected due to the prevailing atmosphere at discharging end (reservoir or aeration fountain or inlet channel), changes from down surge to normal H (static), and travels towards the pump end at speed c. Consequent to reverse flow, NRV at the pump closes, thus disallowing reverse flow which causes pressure rise, i.e., upsurge.

It is thus seen that at T = 0, down surge occurs, and at T = 2L/c, upsurge occurs causing surge pressure rise at the pump end. This wave now travels towards discharging end where it gets reflected again at T = 3L/c and pressure reduces to normal H. The surge wave further travels towards the pump end and reaches the pump end at T = 4L/c; thus, completing a full cycle. The pressure wave keeps on traveling in a cyclic manner till it dies due to friction in pipe surface and water.

The magnitudes of the first down surge and first upsurge are maximum and are, therefore, focus points for analysis without protection and selection of water hammer protection device or multiple devices and analysis with the device(s). Figure 6.25 shows maximum and minimum surge gradients without and with protection devices. It is seen from the figure that sub-atmospheric pressures occur at two locations under no protection case and maximum pressure is very high. With a surge protection device, the sub-atmospheric pressures at both locations are prevented and maximum pressure is also reduced.

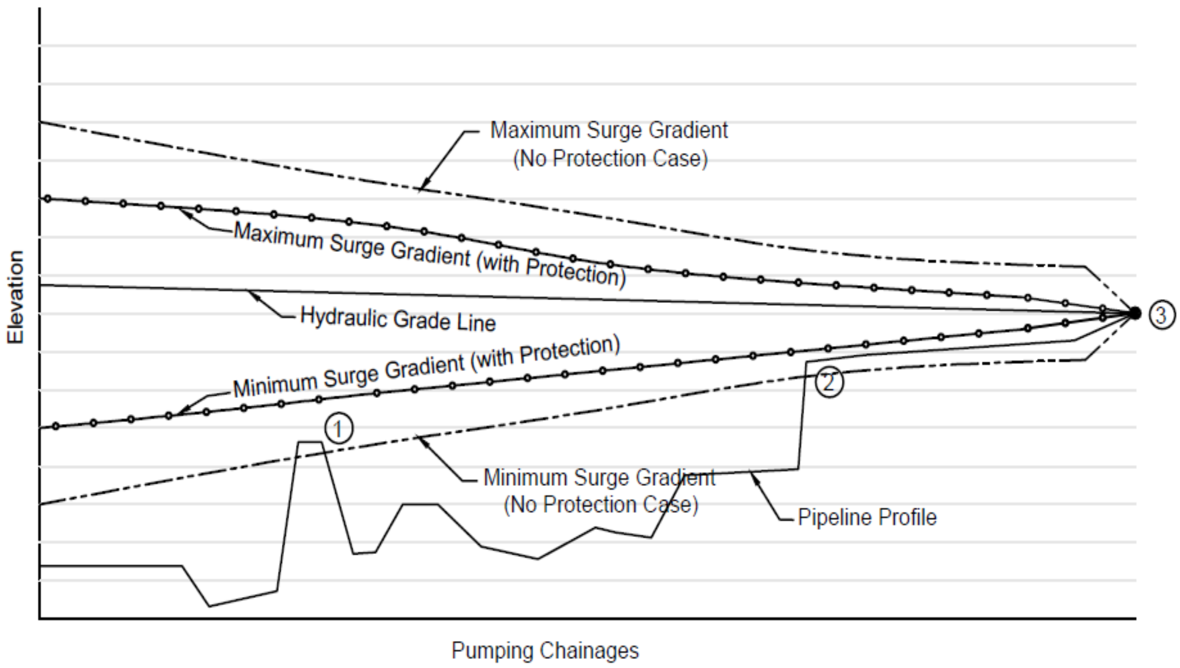


Figure 6.25: Pipeline Profile and Maximum and Minimum Surge Gradients without and with Protection

Note:

1) Peak 1 and pipeline section 2-3 are subjected to Sub-Atmospheric pressure without protection.
2) Due to protection, min surge gradient is above peak and hump section preventing sub-atmospheric pressures.

Both down surge and upsurge cause severe impact on the pipeline as follows:

(a) Down surge

- During down surge, minimum pressure H_{min} shall be equal to H_o-$h_{srg.}$
- Although down surge always causes a pressure drop, the minimum pressure may or may not be below atmospheric pressure. In a high head system, H_{min} shall still be above pipeline profile, and thus, sub-atmospheric pressures are not encountered. In the small and medium head scheme, sub-atmospheric pressures are likely to occur.
- If the pressure drops to a level of vapor pressure, the liquid vaporises generally at peaks/humps along the pipeline causing a vapour cavity and thus separating water columns on two sides.
- Pressure cannot fall below vapor pressure. Vapor pressure is usually 0.5 to 0.7 m depending on water temperature. Thus, at mean sea level, minimum pressure shall be -10.3 + 0.7, i.e., -9.6 m.
- Separated water columns travel towards the cavity, cause a collapse of the cavity, and creates a shock pressure rise. The shock pressure rises wave travels on both sides and can cause a burst or rupture of the pipeline.
- if sub-atmospheric pressures occur, air may enter the pipeline through flange gaskets or joint rings damaging the seal/gaskets.

(b) Upsurge

During upsurge, maximum pressure shall be equal to H_o + h_{max}. If the pressure is above design pressure or field test pressure, a burst or rupture of the pipeline may occur.

6.16.7 Surge Phenomenon due to Single Pump Failure

Even if a single pump of the multi-pump installation fails, sudden velocity reduction does not take place in the pumping main. Hence, no problem is likely to be encountered in the pumping main.

However, due to other running pumps, flow occurs in the delivery of failed pump from the header in opposite direction to forward flow from the failed pump. This sudden change in velocity from forward to reverse direction causes serious upsurge or overpressure on delivery piping, non-return valve (NRV), and delivery valve. This phenomenon is a common cause of failure of NRV and delivery valve. Unfortunately, no protection measure is discussed in the literature.

To protect the pipeline, it is necessary to design delivery piping and both NRV and delivery valve such that the body of the valve and pipeline are suitable to withstand maximum pressure on surge occurrence. Here, h_{srg} is to be calculated considering velocity in delivery piping and not steady state velocity in pumping main.

6.16.8 Surge Phenomenon in Gravity Main

When any line valve or particularly control valve at upstream of discharging end is closed, an upsurge or rise in pressure occurs and the wave travels towards the upstream at wave speed c. If the time of closure of the valve exceeds 2L/c where L is the upstream length of gravity main, no problem is encountered. It is, therefore, essential that the time of closure should be gradual, slow, and should be more than 2L/c.

However, pressure rise of moderate magnitude occurs. Normally, gravity main is designed with a factor of safety above the maximum static head, and adhering to the criteria prevents any possibility of failures.

If the time of closure is less than 2L/c, full surge pressure shall develop. This need to be avoided as gravity main is usually not designed for pressure inclusive of surge pressure.

6.16.9 Guidelines for Design of Pumping Main with and without Surge Protection

a) Without a surge protection device

- The pipeline should be designed to withstand vacuum if encountered and/or maximum design pressure as the sum of operating/working pressure or maximum static head (usually encountered at dip point), whichever is higher, and surge pressure.
- The field test pressure of the pipeline should not be less than the maximum design pressure.
- Shell thickness should be adequate to withstand maximum vacuum with adequate factor of safety.

If the sum of the normal operating pressure and surge pressure exceeds the design pressure, surge protection is essential. Similarly, if a sub-atmospheric pressure occurs or water column separation takes place, surge protection is essential.

b) With surge protection device

Even if a surge protection device is provided, it is normally advisable to design a pipeline to withstand design pressure inclusive of surge pressure. The dependability of surge protection devices should not be taken 100%.

6.16.10 Strategy for Water Hammer Prevention/Protection of Pumping Main

6.16.10.1 Approaches for Strategy and Available Options

To prevent surge phenomenon by converting the pumping main for maximum length or if feasible full length by introducing break pressure tank (BPT) wherever location for BPT is available at suitable elevation, generally not exceeding 25 m. (Refer Figure 6.26).

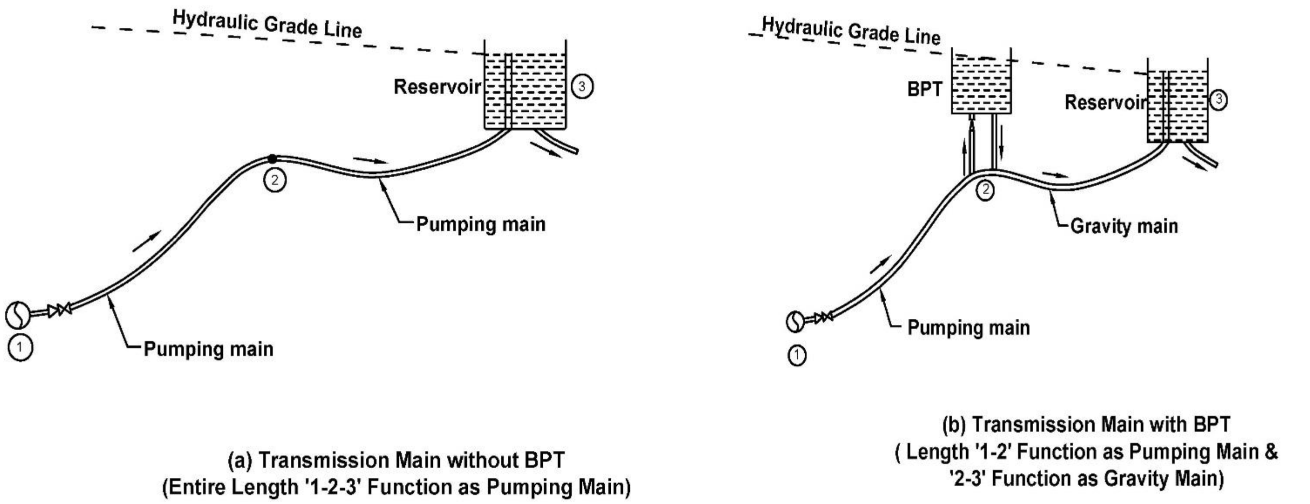


Figure 6.26: Conversion of pumping main into partly pumping main and partly gravity main due to BPT

As evident from the discussion, BPT is not a surge protection device. But if provided, it converts downstream pipeline into gravity main, thus practically free from surge pressures. This also reduces the class/rating of the pipeline resulting in a significant saving.

In the pumping system, if a nearby hillock or high ground is available such that HGL is above GL within 20-25 m, BPT can be introduced at the location. Many such pipeline BPT systems are functioning in the country.

For design aspects, refer discussion for BPT in the earlier section of this Chapter.

To provide water hammer protection device either as a single device or multiple complementary devices in series.

Commonly used water hammer protection devices are as under:

(i) Surge tank/surge shaft
(ii) One way surge tank (also called Discharge tank or Feed tank)
(iii) Two-way surge tank
(iv) Air vessel (Air chamber)-Compressor, bladder, and hybrid types
(v) Surge anticipation valve (also called surge suppressor or surge release valve)
(vi) Pressure relief valve
(vii) Zero velocity valve (ZVV)
(viii) Air cushion valve (ACV)
(ix) Standpipe

Unconventional methods or devices which are not commonly used:

i) Bypass to pumps
ii) Increasing inertia of pump motor set by providing larger flywheel
iii) In-line or intermittent NRV

The essential requirement of NRV in pipeline upstream of location/connection from the surge protection device.

Surge protection devices are provided exclusively for the protection of the pumping main. To make it function effectively, individual delivery pipes, valves, and header need to be isolated by providing NRV/DPCV in the pipeline upstream of the connection tee from the protection device, but downstream of the header. This is also necessary to rely on functioning on common NRV/DPCV instead of individual pump NRVs which may not close simultaneously.

Steps for surge analysis

Surge analysis shall be carried out in the following two steps:

i. Surge analysis for no protection case
ii. Surge analysis with protection device or devices

6.16.10.2 Principles for design and functioning of protection devices

a) Many devices are designed on principle to supply flow into the pipeline as soon as the power fails so as to reduce down surge, prevent sub-atmospheric pressure, and fill the cavity. The upsurge is then correspondingly reduced or may even be entirely eliminated. This is applicable for devices (i), (ii), (iii), and (iv). All the above devices are connected to the pumping main by a tee at the pipeline.

   The devices (iii) and (iv) also control upsurge by throttling reverse flow.

b) The devices (v) and (vi) releases water out from the pipeline; thus, releasing pressure during the upsurge. The devices are fitted on tee branches of the pipeline.
c) The principle behind the design of a zero-velocity valve (ZVV) in (vii), is to arrest forward moving water column when forward velocity is zero and before any return velocity is established. The valve is fitted in-line.
d) The air cushion valve in (viii) as above admits air into the pipeline so as to reduce down surge and fill the cavity. When flow reversal takes place, exit of the air is controlled.

6.16.11 Surge Tank

The device can be selected if HGL at the pumping station is within 20-25 m above the ground level. Refer to Figure 6.27 for the surge tank and surge shaft. The top of the tank is kept above HGL by 1-2 m margins to allow for water level rise during reverse flow or upsurge such as to avoid spilling over. A single connecting pipe from the bottom of the tank is connected to the pumping main with a tee connection. The isolation valve is provided on connecting pipe. The diameter of connecting pipe is usually D/2 where D is the diameter of the pumping main. Only a single pipe connection is required. Preferably the surge tank shall be in RCC construction with dome on top.

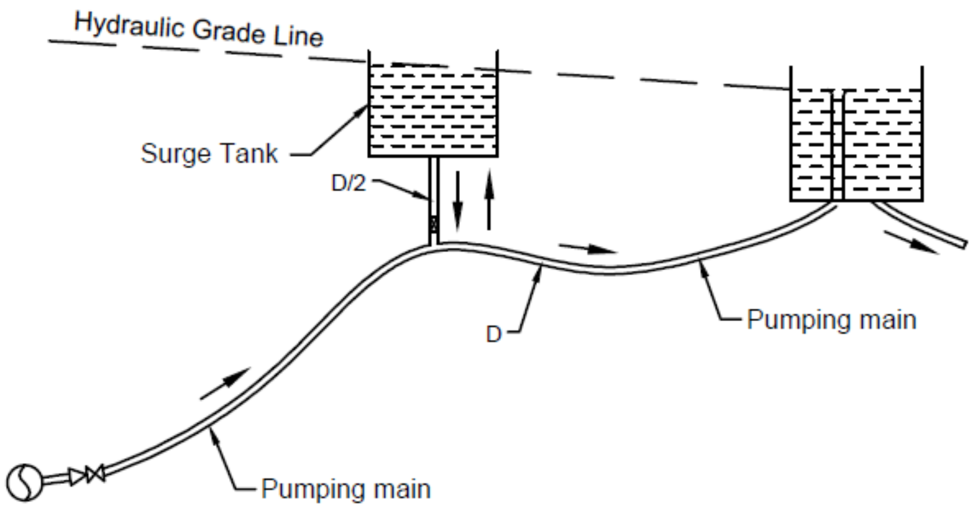


Figure 6.27: Surge Tank

The device is an ideal device amongst all protection devices and should be selected wherever feasible. It is totally without any control, without any moving part or appurtenance, and is maintenance-free.

The design aspect is that during normal operation, the water level (WL) in the tank is at a steady state of WL. The moment power fails and low-pressure wave starts, flow from the tank/shaft passes to the pumping main reducing the rate of change of flow velocity and thus, reducing down surge and if occurred, filling vapour cavity. Due to outflow, WL in the tank reduces. The cross-sectional area of the tank shall be such as to prevent complete draining; rather, 1-2 m water column needs to be retained. The flow from the device converts elastic phenomenon into slow motion phenomenon. During flow reversal, water enters the tank/shaft causing WL to rise.

The amplitude of down surge in the tank/shaft is approximately given by:

$$\text{Amdn} = Vo\sqrt{\frac{Ap \times L}{g \times A_t}} \quad (6.17)$$

Where Amdn = amplitude of down surge
V_o = steady state velocity
A_p = cross-sectional area of pipeline
A_t = cross-sectional area of a surge tank or surge shaft

The approximate area or diameter calculated as per the above equation shall be rechecked by numerical analysis for unsteady flow.

6.16.12 Surge Shaft

The entire discussion for surge tank including location, connecting pipe, design equation, and method are applicable for surge shaft. Margin above HGL and below lowest water level during down surge shall be 2-3 m. The shaft can be in MS construction. Stays are required to counter the effect of wind as shown in Figure 6.28.

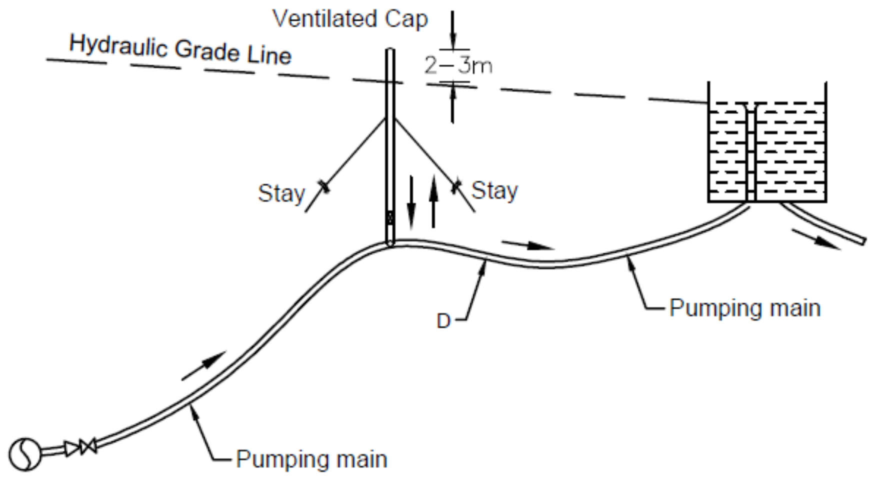


Figure 6.28: Surge Shaft

6.16.13 One-way Surge Tank (Discharge tank / Feed tank)

a) One-way surge tank is located below HGL at maximum up to 25 m height above ground level and connected to pumping main at tee branch. The diameter of connecting pipe is normally D/2 where D is the diameter of the pumping main. Figure 6.29 shows the general arrangement.

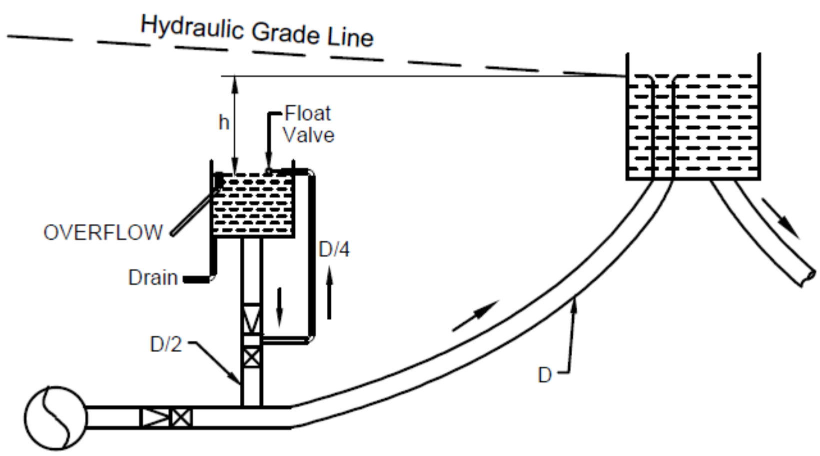


Figure 6.29: One-way Surge Tank

The limitation for the application of the device is that it can function only if $h_{srg} >> h$ (where h is the static head-on tank with reference to discharging end level or subsequent second device).

One-way surge tank functions to reduce down surge; but has no direct capability to reduce to upsurge as under:

When the water level (WL) in the tank is at maximum WL/ TWL, the float valve closes, stopping inflow. The tank during normal operation is at standstill without any inflow or outflow. Following a power failure, velocity in the pipeline starts reducing causing down surge and dropping pressure in the pipeline. When the transitional surge gradient causes pressure below TWL in one-way surge tank, flow from this tank passes through NRV into the pipeline, thus reducing down surge.

Methods for determining the capacity of a one-way surge tank are given by the following two authors.

i) Professor D. Stephenson in the book 'Pipeline Design for Water Engineers' Quantity discharged is given by, Qty = $\frac{A_p \times L \times Vo^2}{2 \times g \times h}$ (6.18)

Where:

Ap = cross-sectional area of pipeline
L = Length of pipeline from tank connection to discharging end or second device
Vo = Steady state velocity
H = static head above one-way surge tank (from tank to the discharging end or next tank)

ii) As per Thorley (1994)

Xs = $\frac{L \times Vo^2}{2 \times g \times h_L}$

Ln = $\left(1 + \frac{h_L}{S}\right)$ (6.19)

Where, Xs = distance travelled by liquid column in coming to rest, Vo = steady state velocity, h_L = friction in length L of pipeline, L = Length from one-way surge tank connection to pipeline end or next tank, S = Static head from tank WL to discharging end or next tank

Normally, safety margin of 25%-50% is desirable in the capacity of a one-way surge tank.

As stated above, a one-way surge tank can reduce down surge, but has no direct capability to reduce upsurge. However, if down surge reduces, consequently, upsurge also reduces. Professor Stephenson has illustrated magnitudes in the chart which gives the following conclusions.

- If h/h_{srg} is 0.5, upsurge or pressure rise is zero.
- The device is effective up to h/h_{srg} = 0.8 due to which rise in pressure is restricted to 60% of h_{srg}.
- If h/h_{srg} is less than 0.5, upsurge is usually less than 25% of h_{srg}..

Important aspects of the device are:

- Multiple tanks are required for a pipeline having a number of high peaks.
- A serious demerit of the device is that float valve malfunctions cause spilling of water and thus results in wastage of pumped water.
- It is difficult to monitor spillage due to float valve failure and proper functioning of a one-way surge tank if located away from the pumping station or if unmanned.

6.16.14 Two-way Surge Tank

A two-way surge tank which is an improvisation over features of a one-way surge tank. Please refer Figure 6.30(a) for illustration. As shown, the tank is a closed vessel mounted without a float valve, but with a vacuum breaker and air release valve.

Further, improvisations are attempted in Maharashtra successfully, as shown in Figure 6.30(b). Instead of a vacuum breaker, a non-return valve can be provided on the vessel with direction to open to the tank which admits air into the tank during down surge, thus preventing vacuum in the tank and multiple air valves can be provided to expel air on restarting pumps. An NRV and bypass to NRV are provided.

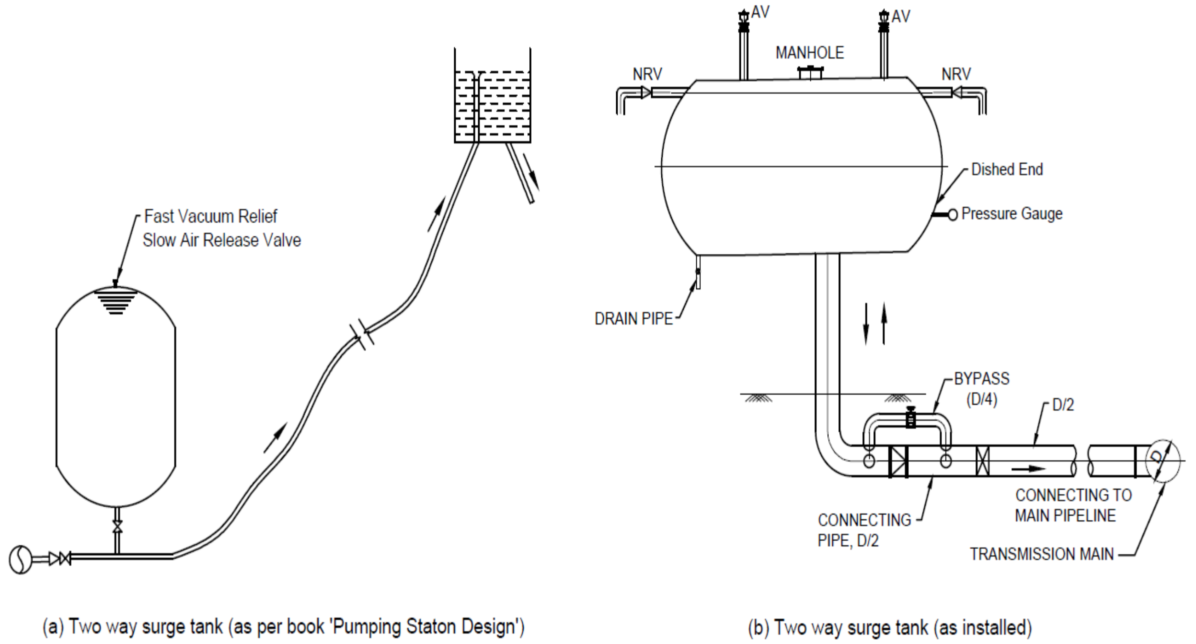


Figure 6.30: Two-Way Surge Tank Installation

The limitation for the application of the device is that it can function only if h_{srg} >> h where h is static head on the tank with reference to discharging end level or second device.

Like a one-way surge tank, a two-way surge tank is located a maximum of up to 20-25 m above ground level. Following a power failure, velocity in the pipeline starts reducing causing a down surge and drop in pressures in the pipeline. When transitional surge gradient causes pressure below the elevation of the two-way surge tank, flow from the tank passes through NRV into the pipeline, thus reducing down surge. During flow reversal, flow passes into the tank through a smaller bypass pipe, thus throttling the inflow and reducing pressure rise. This also helps in releasing the air at a slower rate.

The charts for one-way surge tanks can also be applicable for two-way surge tanks.

6.16.15 Air Vessel (Air Chamber)

Air vessel is a universal protection device and can be designed for any situation of surge phenomenon both down surge and upsurge and any pipeline with or without peaks. Figure 6.31 shows air vessel installation.

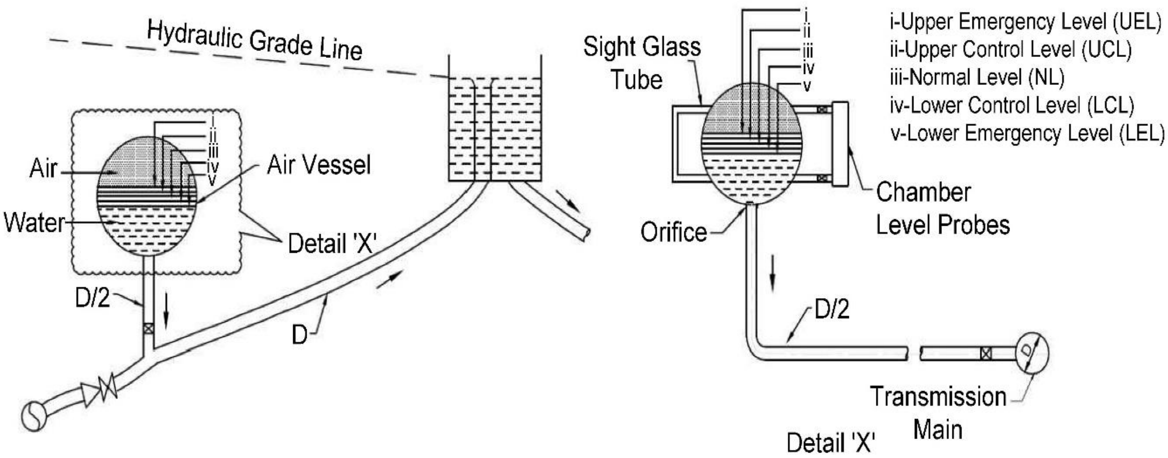


Figure 6.31: Air Vessel Installation

a) Design principle

Design pressure shall be at least equal to the sum of maximum operating pressure and water hammer pressure without any allowance for protection provided. The underlying principle for design pressure is that even if protection is not available for any reason, the air vessel should withstand maximum pressure without any protection and should never fail.

b) Design parameters and functioning

i) Air vessels contain throttling orifice in the outlet. The throttling orifice is designed to achieve minimum head loss during outflow and much higher head loss during inflow.

ii) As the name implies, an air vessel contains water and air on top of the water. The proportion of air and water depends on air vessel design and varies depending on pipeline system characteristics. Important aspects are pipeline parameter and air vessel parameter given by:

Pipeline parameter: $$P_p = \frac{c \times V_o}{g \times H_o^*} \quad (6.20)$$

Air vessel parameter: $$P_a = \frac{2 \times C_o \times c}{Q_o \times L} \quad (6.21)$$

Where, H_o^* = operating pressure in absolute unit, m, C_o = initial air volume in vessel, m^3 and Q_o = steady state discharge, m^3/s

Other values V_o, g, c and L are as defined previously

iii) During down surge, water flows from the vessel to the pipeline, and the space emptied is occupied by the expansion of air in the vessel as per the following equation,

$$H_o^* C_0^{1.2} = H_{min}^* C^{1.2} \quad (6.22)$$

Where:

H_{min}^* = minimum pressure occurred due to down surge in the absolute unit and C = expanded air volume

(Some investigators use the exponent as 1.3 instead of 1.2. However, 1.2 is more common).

The capacity of the air vessel shall be greater than the expanded air volume to ensure that the air vessel is not fully emptied, and air does not enter the pipeline. Normally, about 20%-25% excess volume is kept in design.

Another aspect is that part of air gets dissolved in water causing variation in air-water interface level. Normally, below five levels are kept in interface level bands.

- Normal design interface level at the centre
- Upper interface design level
- Upper emergency interface level (alarm sounds)
- Lower interface design level
- Lower emergency level (alarm sounds)

Normally 25-30 mm spacing is kept in two adjacent levels.

c) Design of air vessel

i) A number of software are available for the design of air vessels. Some widely used software are a) SAP2 developed by the Indian Institute of Science, Bangalore, and b) Surge software Surge: 2018 developed by Kentucky University, USA

ii) The design of an air vessel is quite complex and should be done carefully with the aid of software or charts in books.

d) The necessity of a compressor and air receiver

As discussed, a small portion of air in the vessel gets dissolved in water and therefore, needs to be replenished. Hence, air compressors, generally 1 (working) + 1 (standby) are necessary. To avoid frequent starting and stopping of the air compressor, pressurised air is stored in the air receiver and with level control probes and sensors, the air is fed to the vessel through the air receiver.

Generally, discharge of the air compressor should be adequate to charge the air vessel in a reasonable time of 20-30 minutes.

Air compressor loses pressure building capacity by operation over years. However, the pressure developed by the air compressor should essentially be greater than the operating pressure. In order to allow for the reduction in pressure building capacity, initially specified pressure should be 10%-20% higher than normal operating pressure.

e) Bladder-type air vessel (No compressor is necessary)

In order to avoid the necessity of an air compressor, bladder-type air vessels are developed. The bladder contains compressed air filled corresponding to the required volume C_o at maximum operating pressure. The bladder is a flexible element and stretches to accommodate expanded air volume C. During an upsurge air gets compressed, and the bladder can shrink to hold lower air volume on the overpressure.

- The merits of the bladder are that air is not in contact with water. Hence, air volume reduction due to dissolution is prevented and excess capacity air vessels kept 20-25% can be reduced. Further, the bladder vessel requires little or no maintenance.
- The bladder separates water from the air thereby eliminating the need for frequent recharge of compressed air.
- No compressor and no backup generator.
- No additional power requirement for the compressor.
- No mechanical moving parts other than collapsible bladder inside the vessel - little or no downtime.
- Eliminates corrosion of vessel interior as the water is not in contact with air.

- The pipeline may be started almost immediately in case of pump trip following power failure as the bladder gets charged by the line pressure and relatively smaller size vessel does not require much time for charging (gets charged by the time, pumps are running at their rated speed)
- Reduces the high transient pressures generated at the time of normal pump start-up, thereby minimising the start-up times.

However, bladder vessels being relatively new, there is uncertainty about its life and difficulty in monitoring the condition of the bladder. Another demerit is the flexibility of air vessels with compressors for augmented discharge is not available in bladder-type air vessels.

f) Hybrid type air vessel (no compressor, no bladder)

Hybrid type air vessels are provided with a vent on top of the air vessel and a dipping tube. It works similar to a compressor-type air vessel till the pressure drops to atmospheric pressure. The dipping tube controls the closure of the air vent when the tank is filling, and the length of the dipping tube is varied to maintain the desired air volume under normal operating conditions.

g) Connecting pipe from air vessel to pipeline shall generally be D/2 where D is the diameter of the pipeline. Isolating valve (sluice valve/butterfly valve) shall be provided on connecting pipe.

h) Fittings and appurtenances on air vessel

Following fittings and appurtenances are necessary.

i. Safety valve (to release pressure if exceeds beyond preset pressure)
ii. Air release valve (to release air if quantity exceeds)
iii. Drain valve (air vessel is usually installed at 2-3° inclination to facilitate draining)
iv. Level probes and sight glass tube (to monitor air-water interface levels)
v. Pressure gauge

Merits and demerits of air vessel

The merit of the air vessel is that it is the most universal device suitable for any surge phenomenon and any nature of pipeline.

However, the demerits of the device are:

- It is the costliest device compared to other protection devices.
- It requires considerable maintenance particularly air compressor, level probes, and sight glass tube.
- If the air-water interface level is not maintained, the effectiveness of the air vessel reduces seriously.
- If the air compressor losses pressure building capacity compared to operating pressure, air feeding to the vessel cannot be done rendering the device inoperative.
- In the case of a bladder-type vessel, monitoring the condition of the bladder is very difficult. If provided, its condition should be checked physically as periodical maintenance by taking shut down and entering into air vessel.

An illustrative example of design of air vessel is enclosed in **Annexure 6.9**.

6.16.16 Surge Anticipation Valve

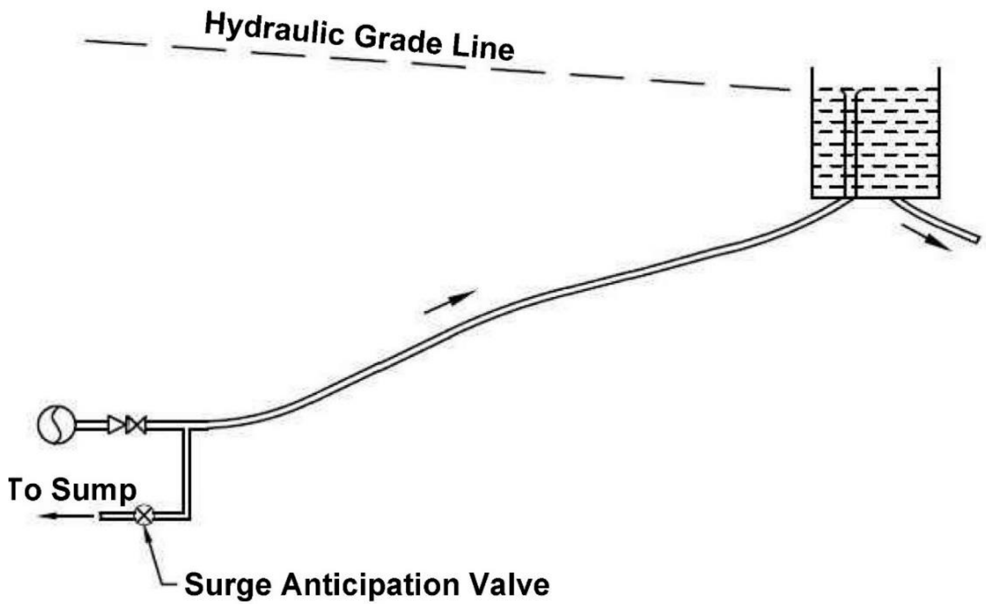


Figure 6.32: General Arrangement of Installation

The valves (Figure 6.32) have actuators that automatically open after sensing pressure drop or electrical fault following power failure and close at a slower rate after discharging water, thereby releasing high pressure. The water release is throttled to avoid cavitation and erosion due to high outflow velocity. The valve offers protection against pressure rise (upsurge) and has no capability to reduce down surge. The valves are generally installed downstream of the delivery valve and its discharge is conveyed to the suction sump.

Demerits of the valve are:

- Pressure release cannot be controlled within practical accuracy.
- The pilot valve may get clogged due to impurities and suspended matters in raw water.
- If the valve releases water before 2 L/c time, down surge may increase and the possibility of sub-atmospheric pressures or even water column separation is high.
- Cavitation within valve components occurs. Due to high velocity, discharging elements are subjected to erosion.
- Due to time delay in the opening of the valves (about five seconds), they are not suitable for a short pipeline less than 2 km length.
- Preventive maintenance is essential for sensing and operating system.

The merit of the valve is that no control is required for its functioning.

The valves are not advisable for important installations, particularly for raw water pipeline systems.

6.16.17 Spring Loaded Pressure Relief Valve

The valve is set to open and release pressure when the pressure reaches the set value. However, no control can be exercised. The valve being exposed to the atmosphere, the springs in the commercial relief valve get corroded and do not offer desired protection. Its use is restricted to the small pumping system.

6.16.18 Air Cushion Valve (ACV)

This valve can function only if sub-atmospheric pressure is encountered in the pipeline, i.e., if c V_o/g > operating head at ACV location. Particularly, peak locations are vulnerable.

The principle of this valve is to admit large quantities of air in the pumping main during down surge causing sub-atmospheric pressure occurrence or water column separation. During reverse flow, ACV entraps the air, compresses it with the returning column, and expels the air under controlled pressure by air cushioning.

The valve is mounted to a tee-joint on the pumping main at locations where water column separation is likely to occur.

6.16.19 Zero Velocity Valve (ZVV)

The principle behind the design of the valve is to arrest the forward moving water column at zero momentum i.e., when its velocity is zero and before any return velocity is established. The ZVV is fitted in the line. The valve consists of an outer shell and an inner fixed dome leaving a streamlined annular passage for water. A closing disc is mounted on central and peripheral guide rods and is held in the closed position by one or more springs when there is no flow of water. A bypass connects the upstream and downstream sides of the disc. The springs are so designed that the disc remains in a fully open position for a velocity of water equal to 25% of the designed maximum velocity in the pipeline.

When the forward velocity becomes less than 25% of the maximum, the disc starts closing at the same rate as the velocity of the water. The disc comes to the fully closed position when forward velocity approaches zero magnitudes, water column on the upstream side of the valve is, thus prevented from acquiring a reversed velocity and taking part in creating surge pressures. The bypass valve maintains balanced pressures on the disc and also avoids vacuum on the downstream side of the valve if that column experiences a certain reversal.

Demerits:

(i) Increase in head loss and significant power cost in the normal operation of the flow system.

Serious demerit of the valve in energy is continuously required during operation due to head loss in ZVV as the valve is installed in the line. (All other devices are on tee branches and therefore, do not cause any addition to pipeline head loss.) The head loss is quite significant due to resistance caused by disc held under spring pressures. The head loss is given by:

$$H_L = \frac{KVo^2}{2g}. \tag{6.23}$$

Here K shall be about 2.5 similar for non-return valve. Thus, if V_o = 1.2 m/s with K = 2.5, H_L = 0.184 m, the discharge Q = 0.943 m^3/s, additional power due to ZVV head loss is 2.15 kW and the energy cost per annum (with pump efficiency = 0.85, motor efficiency = 0.93, tariff = Rs.7 per kWh, 22 h/day operation) shall be Rs.1.21 Lakhs per annum which causes major burden on power cost.

(ii) ZVV is not suitable for raw water pipelines. Floating matters can get entangled in the springs due to which closure of ZVV for protection is seriously compromised.

(iii) Considerable periodical maintenance is required. Periodically, any matters entangled in springs need to be removed physically by opening hand holes.

6.16.20 Standpipe

Figure 6.33 shows the general installation arrangement of the standpipe. The top of the standpipe should be 2-3 m above the HGL at the point of tee connection.

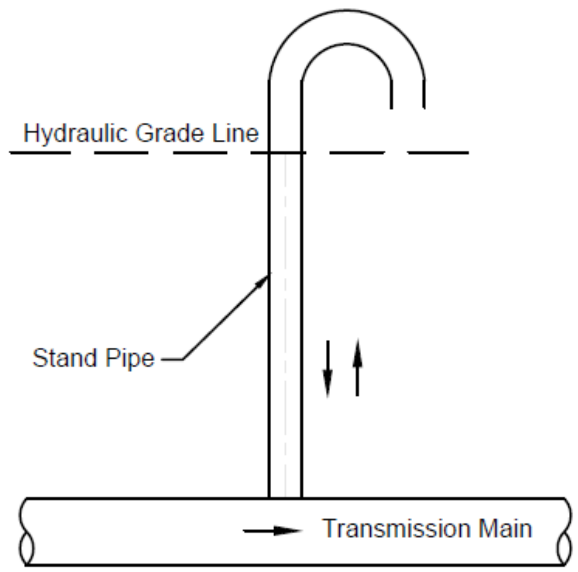


Figure 6.33: General Arrangement of Standpipe

Though the figure shows vertical/upright standpipe, the pipes can also be laid alongside the main pipeline and top terminated 2-3 m above HGL.

The standpipes are generally located at hump/peak which is within 10-12 m of HGL. It prevents sub-atmospheric pressure at tee connection.

6.16.21 Bypass to Low Head Pumps and Booster Pumps

a) This method is applicable for low head pumping systems and also if the pipeline does not have enroute kinks/peaks.

   Figure 6.34 shows a typical installation for a low head system. Subsequent to, power failure, when the pressure in the pipeline falls below WL in the suction tank, water to the pipeline is fed by the bypass line and consequently reduces the down surge. It is important that the diameter of the bypass line shall be of adequate size preferably the same as the pipeline diameter so as to cause very low friction loss.

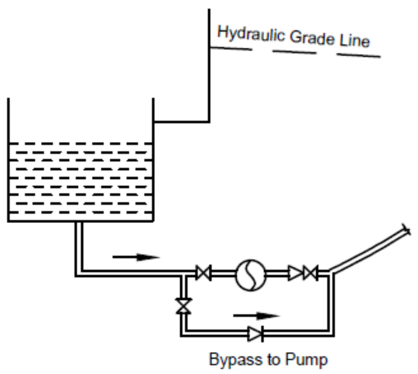


Figure 6.34: A Bypass Line to Low Head Pump

b) This method is very useful for the booster pump system. Figure 6.35 shows the general arrangement. When power failure takes place, the booster pump creates an obstruction to the normal flow resulting in a positive pressure rise on the suction side and pressure drop on the delivery side of the booster. Water flows from the suction side to NRV and further pipelines through the bypass, thus restricting the down surge on the delivery side. The bypass line should preferably be of the same diameter as that of the main pipeline.

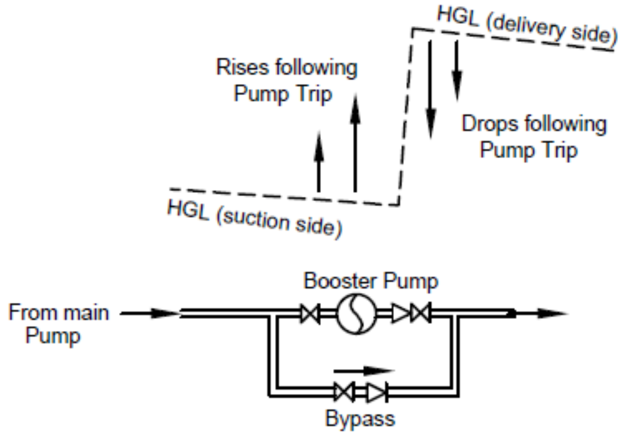


Figure 6.35: Bypass to In-Line Booster Pump Arrangement

6.16.22 Increasing Inertia of Pump Motor Set by Flywheel

The method for calculating pump inertia and motor inertia is described in Chapter 5. Combined inertia of the pump motor set is the sum of pump inertia and motor inertia. A thumb rule is proposed by Professor D. Stephenson in the book 'Pipeline Design for Water Engineers' stating that if $\frac{IN^2}{\rho AL\, h_{srg}^2} > 0.01$, the down surge reduces by about 10%.

Here,

I = moment of inertia of pump motor set in kg.m^2

N = r. p.m.

ρ = fluid density, (1000 kg/m^3 for water)

A = pipeline cross-sectional area in m^2

L = length of pipeline, in m

h_{srg} = surge head

Inertia can be increased by adding a flywheel to the motor shaft. However, practically, it has a limitation as under:

- Motor starting current increases requiring up-gradation of switchgear and motor also. Generally, 100% addition in the moment of inertia may increase the motor cost by 20%-25%.
- Flywheel addition is possible only in the case of a horizontal motor. The flywheel cannot be provided on vertical motor.

Thus, adding a flywheel is generally not advisable except for a short pipeline system, where it may be economical.

6.16.23 Suitability and Compatibility of Devices for Series Installation

a) More than one protection device is required for longer pumping mains, depending on nature and ups-downs of profile and hydraulic characteristics. For large and medium-length pipelines, optimisation can be possible by selecting more than one device to be installed in series. Depending on the length of the pipeline and its profile, even 3-4 devices may be necessary if suitably designed and optimised.

As an example, a very large air vessel was required for over 70 km long pumping main. By introducing a two-way surge tank at enroute hump at 40 km, the capacity of the air vessel was drastically reduced, and the combined cost was half of the cost of a single large air vessel.

It is possible to select devices (Table 6.10) for series installations.

Table 6.10: Devices for series installations

Upstream Choice (Usually at Pumping Station)	Downstream Devices
i) Air vessel	One-way surge tank/two-way surge tank/surge tank/surge shaft/standpipe
ii) Surge tank	One-way surge tank/two-way surge tank/standpipe
iii) One-way surge tank/two-way surge tank	Surge shaft/one-way surge tank/two-way surge tank/standpipe
iv) ZVV and ACV can be used in combination in the pipeline. ACVs are installed at peaks and ZVVs are installed at dip points.	

b) Figure 6.36 illustrates a typical example where the following four protection devices are installed in series.

i) Air vessel adjacent to pumping station
ii) One-way surge tank or two-way surge tank at intermediate peak satisfying condition that $(cV_o/g) > h$ where h is as defined in the section
iii) Surge tank at a pronounced peak where HGL is within 20-25 m above ground level
iv) Standpipe at next pronounced peak where HGL is about 10 m higher than ground level

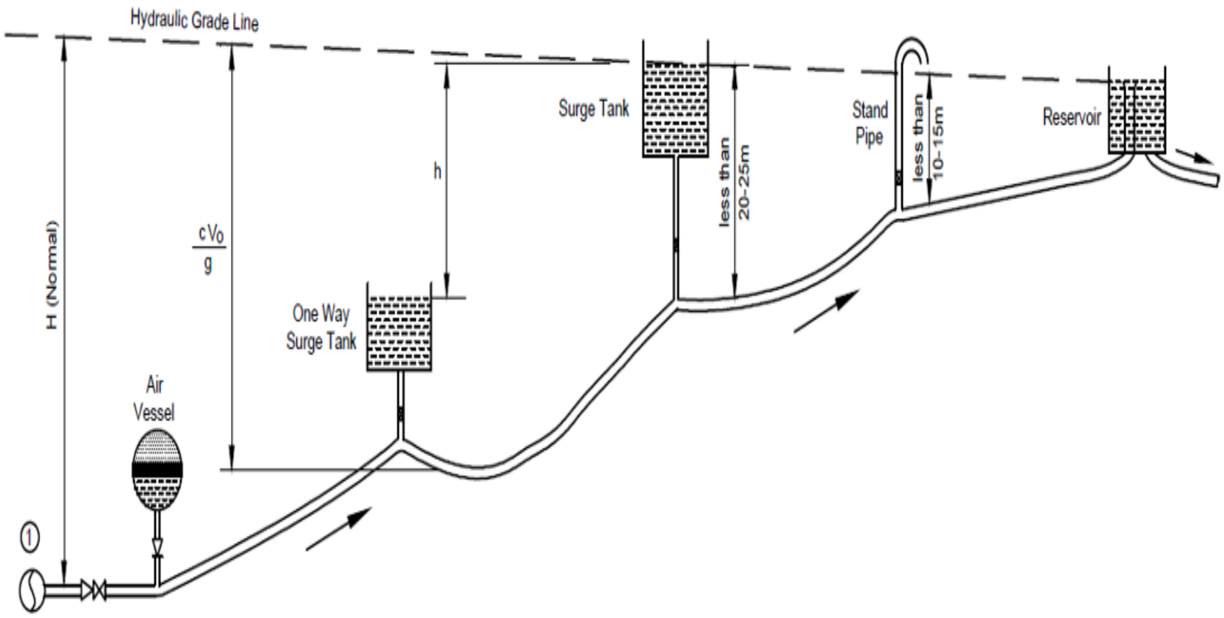


Figure 6.36: Multiple Surge Protection Devices on Very Long Pipeline

c) It is not advisable to provide an air vessel at an intermediate location in combination with other devices at a pumping station due to difficulty in maintaining and controlling interface levels.

6.16.24 In-line Reflux Valve (NRV / DPCV)

Normally, a reflux valve installed on its own in the pipeline will not reduce surge pressures, although it may limit the lateral extent of the shock. In fact, in some situations, indiscriminate positioning of reflux valves in a line could be detrimental to surge pressures. For instance, if a pressure relief valve was installed upstream of the reflux valve, the reflux valve would counteract the effect of the other valve. It may also amplify reflections from branch pipes or collapse vapour pockets.

There are situations where water column separation and the formation of vapour pockets in the pipeline following pump stoppage would be tolerable, provided the vapour pockets did not collapse resulting in surge pressures. Reversal of the water column beyond the vapour pocket could in fact be prevented with an in-line reflux valve at the downstream end of the vapour pocket. The water column would be arrested at its point of minimum momentum, so there would be little head rise.

In locating the reflux valve, allowance should be made for some lateral dispersal of the vapour pocket. The valve should be installed at a suitable dip in the pipeline in order to trap the vapour pocket and to ensure the proper functioning of the valve doors when the water column returns.

A small diameter bypass to the reflux valve should be installed to permit slow refilling of the vapour pocket otherwise overpressures may occur on restarting the pumps. The diameter of the bypass should be of the order of one-tenth of the pipeline diameter. An air release valve should be installed in the pipeline at the peak to release air that would come out of the solution during the period of low pressure.

In-line reflux valves would normally be used in conjunction with a surge tank, one-way surge tank, two-way surge tank, or air vessel where NRV is installed on the upstream side of the tee connection. Following pump shutdown, the tank or vessel would discharge water into the pipe on the downstream side of the reflux valve.

6.16.25 Non-Suitable Devices for Installation in Combination

a) The following combinations are not suitable for series installations:

(i) Air vessel + ACV (as dependability on ACV is not advisable)
(ii) Air vessel + ZVV
(iii) Air vessel + Surge anticipation valve/Pressure relief valve
(iv) Bypass to pump in combination with other devices

b) Reflux valve/NRV/DPCV should not be generally installed in combination with other devices except for the following positions:

If intermittent one-way surge tank(s) or two-way surge tank(s) are installed, it is advisable to install RV/NRV/DPCV upstream of connections to the pipeline for second and subsequent tanks to avoid the following situation.

At some stage after a power failure, second tank (one-way surge tank/two-way surge tank) starts feeding upstream first tank.

6.16.26 Preferred Order for Selection of Devices

On the basis of not only cost but also required maintenance, reliability, and effectiveness of protection, the preferred order for selection can be stated as follows.

Applicable criteria for suitability of the device are restated against each device in Table 6.11 below.

Table 6.11: Preferred Order for Selection of Surge Protection Devices

S. No.	Device/Method	Applicable Conditions	Important Merit/Demerit
i.	**Bypass to pumps**	• $(cV_o/g) > H_o$ • No peak beyond the pumping station • Negligible head loss in bypass	• Very simple, control-free, and maintenance-free • Not suitable for undulating pipeline
ii.	**Surge tank**	• HGL within 20-25 m above ground level	• Very simple, control-free, and maintenance-free; Ideal device
iii.	**Two-way surge tank**	• $(cV_o/g) > h$ • No pronounced peak/hump on downstream pipeline	• Control-free and maintenance-free • Upsurge can be controlled
iv.	**One-way surge tank**	• $(cV_o/g) > h$ • No pronounced peak/hump on downstream pipeline	• Malfunctioning of float valve is a serious handicap in a one-way surge tank causing spilling over and wastage of pumped water and energy • One-way surge tank has no capability for controlling upsurge
v.	**Air vessel**	• Suitable for any pipeline and hydraulic characteristics	• Very costly and needs maintenance

S. No.	Device/Method	Applicable Conditions	Important Merit/Demerit
vi.	**Standpipe**	• HGL within 10-15 m from ground level	• Localised use for avoiding water column separation or sub-atmospheric pressures • No capability to control the upsurge • Spills over during upsurge conditions causing wastage of water and creating an environmental issue
vii.	**Surge anticipation valve (Surge Suppressor/Surge release valve)**	• $(cV_o/g) < H_o$ • Water should be free from floating matters • L > 2000 m	• Not suitable for raw water pipeline • Increases down surge • Need intensive care and maintenance
viii.	**Three stage air valves**	• Suitable for any pipeline	• Usually used in addition to other devices.
viii.	**Pressure relief valve**	• Suitable for short pumping main	• Not dependable
ix.	**ACV**	• $(cV_o/g) > H_o$	• Entire air admitted into the pipeline may not be expelled causing air trapping • Controlling air inflow and air outflow is not within practical accuracy
x.	**ZVV**	-	• Valve is fitted in-line due to which significant head loss takes place causing energy loss and a major burden on operation cost. • No capability for controlling down surge

6.16.27 Surge Phenomenon on Suction Pipes of Pumps

Suction pipes subsystems can be broadly categorised into three (a) normal pumping installations where pumps are above/by the side/near suction sump (b) long suction/inlet pipeline from dam or lake at higher elevation (c) In-line booster pump installation.

a) Normal pump installation - Suction pipe short

Nothing of much consequence will happen where the suction line is short irrespective of suction lift or positive suction. The suction sump shall function both as a surge relief outlet as well as a feeding tank and transient surge pressures will dissipate.

b) Long suction pipe/Inlet pipe

Following pump stoppage due to power failure, the long suction pipe/inlet pipe shall be initially subjected to upsurge and subsequently, down surge.

The solution is to control the upsurge by providing a surge tank or surge shaft so as to divert inflow to the protection device, thereby releasing surge pressure and converting surge phenomenon to slow motion phenomenon. Figure 6.37 shows an arrangement of a surge shaft on a long suction line from the dam. The figure also shows an alternative of a surge tank.

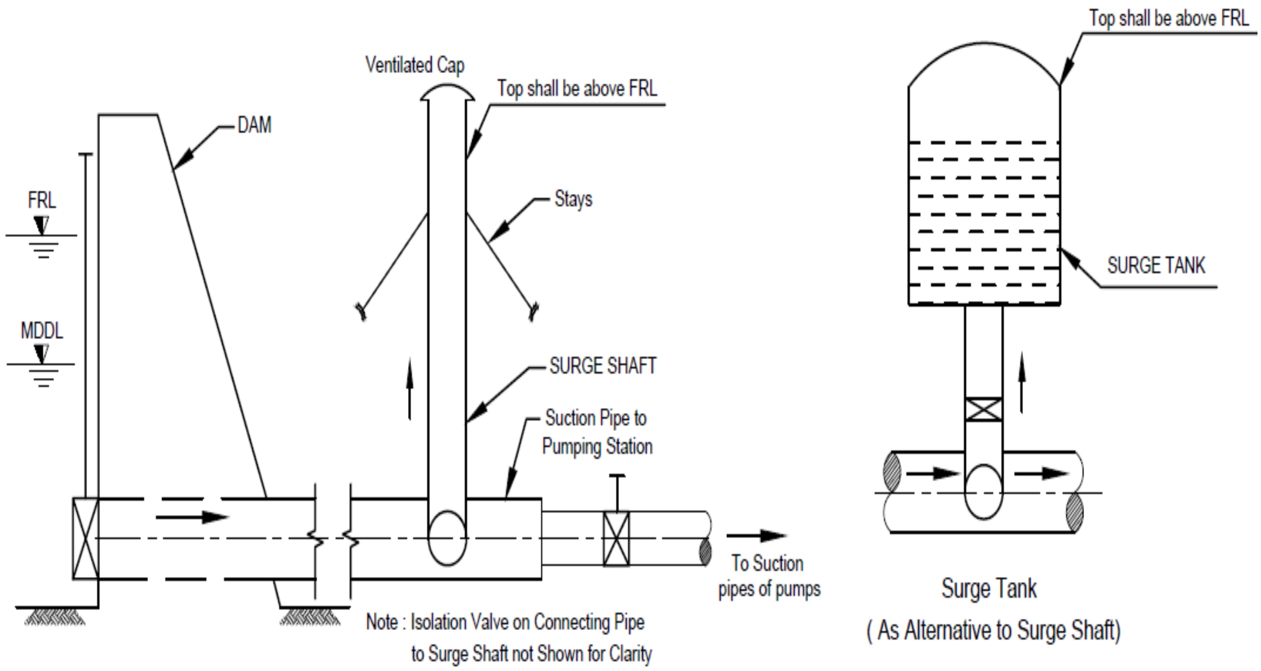


Figure 6.37: Typical Surge Protection Device for Long Suction Pipeline

c) The suction side of the in-line booster pump

Figure 6.37 shows surge occurrence on both the suction side and delivery side of the booster pump. Depending on the pressure on the suction side of the pump, one of the following three types of protection devices can be adopted for protecting the suction pipeline.

i) Surge tank (limit positive pressure on suction side < 20-25 m above ground level
ii) Surge shaft (Same limit as above)
iii) Surge anticipation valve

As for surge protection on the delivery side bypass to a booster, as shown in the figure, it can be provided depending on pipeline characteristics; or if surge pressures are high, other protection devices can be provided after detailed analysis.

CHAPTER 7: WATER QUALITY TESTING AND LABORATORY FACILITIES

7.1 Introduction

Water quality is typically categorised into physical, chemical, microbiological, and radiological parameters. As the water comes into contact with various substances in different phases of the hydrologic cycle, such as rainfall, runoff, infiltration, impoundment, use, and evaporation, many substances are dissolved and suspended. Drinking water quality is influenced by source water, efficacy of water treatment plants, integrity, and condition of the water distribution system, and, as importantly, the condition of service lines to households.

Focused attention on water quality is critical to all design aspects of various components of the water supply system. The raw water quality of source water is an important aspect of the detailed engineering design of the components throughout the system, e.g., intake arrangements, treatment process, boosting/correcting measures needed in the transmission of treated water, for delivering quality water to all the consumers. The design should therefore take into consideration the physical, chemical, biological, and radiological water quality parameters associated with the source water, which may, at present, be within the permissible limits, but could have increasing trends and might change during the design period and make the water unfit for drinking unless specific treatment is installed before it can be delivered to the consumers.

The primary purpose of water quality monitoring in the water supply system is to ensure compliance with water quality criteria and standards stipulated by the concerned agencies, assess the state of water and determine trends. Strategies need to be developed for undertaking monitoring and surveillance, collecting and analysing data and delineating preventive and remedial actions for the provision of safe water, which are explained in detail in Chapter 8: Drinking Water Quality Monitoring and Surveillance of Part B of the Manual.

7.2 Health Effects of Unsafe Drinking Water

Water is necessary for survival, and everyone should have access to an adequate, safe, and reliable water supply. In terms of quantity and quality, water profoundly affects the health and well-being of individuals and the community. Pathogens in drinking water are likely to result in infectious diseases. In addition, chemical contaminants are increasing in the water, posing health risks when their presence is above stipulated standards and consumed without treatment. Many factors, such as the type of contaminant and concentration in water, the quantity of water consumed, duration of exposure and individual susceptibility, largely govern the severity of the impact of the disease. Inadequate management, monitoring and surveillance of urban, industrial, and agricultural water and sanitation services contaminate or chemically pollute the drinking water, exposing individuals to preventable health risks.

Diseases like cholera, diarrhoea, dysentery, hepatitis A, typhoid and polio have been linked to contaminated water and poor sanitation. Tables 7.1 give details of the diseases likely to be caused by chemical contaminants (if exceed the limits stipulated in drinking water standards IS 10500:2012). Table 7.2 provide details of diseases mainly gastrointestinal diseases due to pathogens in drinking water.

Table 7.1: Water Contaminants and Associated Diseases

S. No.	Contaminants	Diseases or Impacts
1	Alkalinity	Gastrointestinal irritation and irritation to the eyes, skin, and mucous membranes

2	Hardness	Eczema, in addition to this it causes scaling and inability to form lather
3	Copper	Vomiting, diarrhoea, stomach cramps, nausea, liver damage, and kidney disease
4	Chloride	Chloride toxicity has only been observed rarely in impaired sodium chloride metabolism cases, such as congestive heart failure
5	Fluoride	Weakness, shallow respiration, spasms and convulsions, jaundice and urine suppression, discoloration of teeth, mottling in infants, and fluorosis
6	Nitrates as NO_3	Methaemoglobinaemia and congenital malformations
7	Sodium	Nausea, vomiting, muscular twitching and rigidity, convulsions, and cerebral and pulmonary oedema
8	Arsenic	Fatigue, nausea, vomiting, stomach pain, bloody diarrhoea, thickening or discoloration of the skin leading to skin cancer, numbness in hands and feet
9	Cadmium	Bone disease, osteomalacia, choking, vomiting, diarrhoea, abdominal pain, anaemia, renal dysfunction
10	Cyanide	Unresponsive hypotension, slow respiration and gasping, cyanosis at high levels and finally, death

(Source: World Health Organisation, 2017)

Table 7.2: Microorganisms and Associated Diseases

S. No.	Microorganism	Diseases or Impacts
Pathogenic protozoans		
1.	*Cryptosporidium hominis, C. parvum*	Gastroenteritis
2.	*Acanthamoeba castellanii*	Amoebic meningoencephalitis
3.	*Entamoeba histolytica*	Amoebic dysentery
4.	*Balantidium coli*	Dysentery
5.	*Giardia lamblia*	Giardiasis (gastroenteritis)
Pathogenic bacteria		
6.	*Shigella spp.*	Bacillary dysentery
7.	*Salmonella typhi*	Typhoid fever
8.	*Salmonella paratyphi*	Paratyphoid fever
9.	*Vibrio cholerae*	Cholera
10.	*Leptospira spp.*	Leptospirosis
11.	*E. coli O157:H7 (Pathogenic E. coli)*	Gastroenteritis, ear and eye infections, diarrhoea, urinary tract infections, skin diseases
Enteric Viruses		
12.	Polio viruses	Poliomyelitis

S. No.	Microorganism	Diseases or Impacts
13.	Rotaviruses	Gastroenteritis
14.	Hepatitis A virus	Infectious hepatitis, jaundice
15.	Hepatitis E virus	Infectious hepatitis, jaundice, miscarriage, and death

(Source: World Health Organisation, 2017)

Legislative Provision

Article 21 of 'The Constitution of India', 1949 states "Protection of life and personal liberty: No person shall be deprived of his life or personal liberty except according to procedure established by law". Thus, the right to access to drinking water is fundamental to life and there is a duty on the State under Article 21 to provide safe and clean drinking water to its citizens.

India is a party to the Resolution of the UNO passed during the United Nations Water Conference in 1977: "All people, whatever their stage of development and their social and economic conditions, have the right to have access to drinking water in quantum and of a quality equal to their basic needs."

The United Nations (UN) has determined that access to clean water and sanitation facilities is a fundamental human right. Sustainable Development Goal 6 is about "clean water and sanitation for all".

7.3 Standards and Guidelines

There are various standards and guidelines published by the Bureau of Indian Standards (BIS), Ministry of Environment, Forests and Climate Change (including Central Pollution Control Board) and Ministry of Jal Shakti from time to time, which form the basis of planning the Water Quality Testing requirements and laboratory facilities to be provided, to provide safe drinking water to all residents, at all times.
The Standards and Guidelines applicable are:

1. IS 10500: 2012 Drinking Water - Specifications (second revision)
2. IS 3025: 2019 Methods of Sampling and Test (Physical and Chemical) for Water and Wastewater
3. CPCB Water Quality Standards - Designated Best Use Water Quality Criteria
4. Drinking Water Quality Monitoring & Surveillance Framework, Jal Jeevan Mission, Ministry of Jal Shakti, October 2021

7.4 Water Quality Regulations

Water that is fit for human consumption (drinking water) must meet specific criteria/guidelines values/standards to avoid adverse health effects as mentioned in the above paragraphs. For design purposes, contaminants that can be treated with the same technique are usually grouped. In most cases, it is impossible to address each contaminant in terms of treatment. However, specific contaminants (e.g., fluoride, arsenic, iron etc.) must be removed and require specific treatment.

7.4.1 Raw Water Quality Criteria

The criteria applicable for the surface water source, are prescribed by Central Pollution Control Board (CPCB) as 'Water Quality Criteria' which are based on the designated use. The water quality criteria are stipulated to define the goals and aspirations for the usage of each water body. Typical examples of designated uses include fish and wildlife conservation and propagation, recreation, water supply (as source) to the general public, agricultural, industrial, navigational, and other applications.

Water quality criteria of CPCB, based on designated use, are given in Table 7.3, which indicates that criteria provided against Class A and Class C apply to water supply schemes with surface water sources. ULBs can use criteria A and C for selecting appropriate surface water source for water supply schemes.

Table 7.3: Surface water quality criteria for designated best use

Designated Best Use	Class of water	Criteria
Drinking water source without conventional treatment but after disinfection	A	• Total coliforms organism MPN/100ml shall be 50 or less • pH between 6.5 and 8.5 • Dissolved oxygen 6 mg/L or more • Biochemical oxygen demand five days 20 °C, 2 mg/L or less
Outdoor bathing (organized)	B	• Total coliforms organism MPN/100ml shall be 500 or less • pH between 6.5 and 8.5 • Dissolved oxygen 5 mg/L or more • Biochemical oxygen demand five days 20°C, 3 mg/L or less
Drinking water source after conventional treatment and disinfection	C	• Total coliforms organism MPN/100 ml shall be 5,000 or less • pH between 6 to 9 • Dissolved oxygen 4 mg/L or more • Biochemical oxygen demand five days 20°C, 3 mg/L or less
Propagation of wildlife and fisheries	D	• pH between 6.5 to 8.5 • Dissolved oxygen 4 mg/L or more • Free ammonia (as N) 1.2 mg/L or less
Irrigation, industrial cooling, controlled waste disposal	E	• pH between 6.0 to 8.5 • Electrical conductivity at 25°C micro mhos/cm Max.2250 • Sodium absorption ratio max. 26 • Boron max. 2 mg/L

(Source: Central Pollution Control Board guidelines, 2019)

7.4.2 Drinking Water Specification (IS 10500:2012)

Water, an excellent solvent, ensures the solubility of chemicals from natural and anthropogenic sources. There can be several constituents in water which may adversely affect water quality. In providing safe drinking water, determining the water quality parameters is essential to avoid adverse health effects on consumption. All the water quality constituents (parameters) do not have adverse health effects and their determination has implications ranging from water treatment to aesthetic value.

The Bureau of Indian Standards (BIS) prescribes the tests for assessing water quality and standards that must be met for making the water potable in IS 10500:2012 and subsequent amendments and IS 3025:2019 - Methods of Sampling and Test (Physical and Chemical) for Water and Wastewater.

In the absence of an alternate source, the standard specifies acceptable and permissible limits. It is recommended that the acceptable limit be implemented, as values above those listed under 'Acceptable' render the water unfit for consumption. Such a value may be tolerated in the absence of an alternative source. However, if the value exceeds the 'permissible limit in the absence of an alternate source' limits, the water must be rejected for drinking.

IS 10500:2012 - Drinking Water - Specification has the following categories of water quality parameters for which limits (standards) are provided:

i) Organoleptic and physical parameters
ii) General parameters concerning substances undesirable in excessive amounts (chemical parameters)
iii) Parameters concerning toxic substances (chemical parameters)
iv) Pesticide residues (chemical parameters)
v) Parameters concerning radioactive substances
vi) Microbiological parameters namely indicator bacteria, viruses, protozoa, and helminths
vii) Biological parameters namely algae, zooplankton, flagellates

Drinking water shall comply with the requirements given in Tables 7.4 to Table 7.9. The methods of sampling and testing have been explained in relevant parts of IS 3025.

It is further mentioned that water quality parameters once introduced by BIS as part of IS 10500:2012 should be mandatorily monitored by ULBs. There are emerging contaminants such as pharmaceutical and personal care products (PPCPs), persistent organic pollutants (POPs), per and poly fluoroalkyl substances (PFAS), etc., which also have health concerns, and are being monitored in developed countries. Monitoring these emerging contaminants will also be mandatory once they are part of IS 10500:2012. It is suggested to large and medium ULBs to collate data of emerging contaminants for water sources if available through reliable references such as research and academic institutions.

Table 7.4: Organoleptic and Physical Parameters

S. No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to Part of IS 3025	Remarks
1	2	3	4	5	6
i)	Colour, Hazen units, Max	5	15	Part 4	Extended to 15 only, if toxic substances are not suspected in absence of alternate sources
ii)	Odour	Agreeable	Agreeable	Part 5	a) Test cold and when heated b) Test at several dilutions
iii)	pH value	6.5-8.5	No relaxation	Part 11	-

S. No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to Part of IS 3025	Remarks
iv)	Taste	Agreeable	Agreeable	Parts 7 and 8	Test to be conducted only after safety has been established
v)	Turbidity, NTU, Max	1	5	Part 10	-
vi)	Total dissolved solids, mg/L, Max	500	2000	Part 16	-

NOTE - It is recommended that the acceptable limit is to be implemented. Values in excess of those mentioned under "acceptable" render the water not suitable, but still may be tolerated in the absence of an alternative source but up to the limits indicated under permissible limit in the absence of alternate source in col. 4, above which the sources will have to be rejected.

Table 7.5: General Parameters Concerning Substances Undesirable in Excessive Amounts

S. No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate source	Method of Test, Ref to	Remarks
1	*2*	*3*	*4*	*5*	*6*
i)	Aluminium (as Al), mg/L, Max	0.03	0.2	IS 3025 (Part 55)	-
ii)	Ammonia (as total ammonia N), mg/L Max	0.5	No relaxation	IS 3025 (Part 34)	-
iii)	Anionic detergents (as MBAS) mg/L, Max	0.2	1	Annex K of IS 13428	-
iv)	Barium (as Ba), mg/L, Max	0.7	No relaxation	Annex F of IS 13428* or IS 15302	-
v)	Boron (as B, mg/L, Max	0.5	2.4**	IS 3025 (Part 57)	Permissible value changed from 1 to 2.4 mg/L
vi)	Calcium (as Ca), mg/L, Max	75	200	IS 3025 (Part 40)	-
vii)	Chloramines (as Cl_2), mg/L, Max	4	No relaxation	IS 3025 (Part 26)* or APHA 4500-Cl G	Chloramines are used as disinfectant and a minimum residual level should be maintained but it

S. No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate source	Method of Test, Ref to	Remarks
1	*2*	*3*	*4*	*5*	*6*
					should be ensured that maximum level doesn't exceed 4.0 mg/L
viii)	Chloride (as Cl), mg/L, Max	250	1000	IS 3025 (Part 32)	-
ix)	Copper (as Cu), mg/L, Max	0.05	1.5	IS 3025 (Part 42)	-
x)	Fluoride (as F), mg/L, Max	1	1.5	IS 3025 (Part 60)	-
xi)	Free residual chlorine, mg/L, Min	0.2	1	IS 3025 (Part 26)	Chlorine is used as disinfectant. This is applicable only when water is chlorinated. Tested at consumer end. When protection against viral infection is required, it should be minimum 0.5 mg/L.
xii)	Iron (as Fe), mg/L, Max	1**	No relaxation	IS 3025 (Part 53)	Total concentration of manganese (as Mn) and iron (Fe) shall not exceed 1 mg/L which was increased from 0.3 mg/L.
xiii)	Magnesium (as Mg), mg/L, Max	30	100	IS 3025 (Part 46)	-
xiv)	Manganese (as Mn), mg/L, Max	0.1	0.3	IS 3025 (Part 59)	Total concentration of manganese (as Mn) and iron (Fe) shall not exceed 0.3 mg/L
xv)	Mineral oil, mg/L, Max	1**	No relaxation	Clause 6 of IS 3025 (Part 39) Infrared partition method	Permissible value increased from 0.5 to 1mg/L
xvi)	Nitrate (as NO_3), mg/L, Max	45	No relaxation	IS 3025 (Part 34)	-
xvii)	Phenolic compounds (as	0.001	0.002	IS 3025 (Part 43)	-

S. No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate source	Method of Test, Ref to	Remarks
1	*2*	*3*	*4*	*5*	*6*
	C_6H_5OH), mg/L, Max				
xviii)	Selenium (as Se), mg/L, Max	0.01	No relaxation	IS 3025 (Part 56) or IS 15303*	-
xix)	Silver (as Ag), mg/L, Max	0.1	No relaxation	Annex J of IS 13428	-
xx)	Sulphate (as SO_4), mg/L, Max	200	400	IS 3025 (Part 24)	May be extended to 400 provided that Magnesium do not exceed 30
xxi)	Sulphide (as H_2S), mg/L, Max	0.05	No relaxation	IS 3025 (Part 29)	-
xxii)	Total alkalinity as calcium carbonate, mg/L, Max	200	600	IS 3025 (Part 23)	-
xxiii)	Total hardness (as $CaCO_3$), mg/L, Max	200	600	IS 3025 (Part 21)	-
xxiv)	Zinc (as Zn), mg/L, Max	5	15	IS 3025 (Part 49)	-

NOTES:

1. Approved and validated international test methods from ISO/ APHA/ ASTM/ AOAC/ EPA/ EN may also be followed.
2. In case of dispute, methods given at column 5 and wherever indicated by '*'shall be the referee method. **as per latest amendments.
3. It is recommended that the acceptable limit is to be implemented. Values in excess of those mentioned under 'acceptable' render the water not suitable, but still may be tolerated in the absence or an alternative source but up to the limits indicated under 'permissible' limit in the absence or alternate source' in col. 4, above which the sources will have to be rejected.

Table 7.6: Parameters Concerning Toxic Substances

S. No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to	Remarks
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
i)	Cadmium (as Cd), mg/L, Max	0.003	No relaxation	IS 3025 (Part 41)	-
ii)	Cyanide (as CN), mg/L, Max	0.05	No relaxation	IS 3025 (Part 27)	-
iii)	Lead (as Pb), mg/L, Max	0.01	No relaxation	IS 3025 (Part 47)	-
iv)	Mercury (as Hg), mg/L, Max	0.001	No relaxation	IS 3025 (Part 48)/Mercury analyser	-
v)	Molybdenum (as Mo), mg/L, Max	0.07	No relaxation	IS 3025 (Part 2)	-
vi)	Nickel (as Ni), mg/L, Max	0.02	No relaxation	IS 3025 (Part 54)	-
vii)	Pesticides, µg/L, Max	See Table 7.8	No relaxation	See Table 7.8	-
viii)	Polychlorinated biphenyls, mg/L, Max	0.0005	No relaxation	ASTM 5175* Or APHA 6630	-
ix)	Polynuclear aromatic hydrocarbons (as PAH), mg/L, Max	0.0001	No relaxation	APHA 6440	-
x)	Total arsenic (as As), mg/L, Max	0.01	No relaxation**	IS 3025 (Part 37)	Permissible value changed from 0.05 to no relaxation
xi)	Total chromium (as Cr), mg/L, Max	0.05	No relaxation	IS 3025 (Part 52)	-
xii)	Trihalomethanes:				
	a) Bromoform, mg/L, Max	0.1	No relaxation	ASTM D 3973-85* Or APHA 6232	-
	b) Dibromochloromethane, mg/L, Max	0.1	No relaxation	ASTM D 3973-85* Or APHA 6232	-
	c) Bromodichloromethane, mg/L, Max	0.06	No relaxation		-
	d) Chloroform, mg/L, Max				

S. No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to	Remarks
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*
		0.2	No relaxation	ASTM D 3973-85* Or APHA 6232 ASTM D 3973-85* or APHA 6232	-
xiii)	Uranium, mg/L, max	0.3***	No relaxation	IS 3025 (Part 65*) or IS 14194 (Part 3)	***New addition

NOTES:

1. Approved and validated international test methods from ISO/ APHA/ ASTM/ AOAC/ EPA/ EN may also be followed.
2. In case of dispute, methods given at column 5 and wherever indicated by '*'shall be the referee method. **as per latest amendments.
3. It is recommended that the acceptable limit is to be implemented. Values in excess of those mentioned under 'acceptable' render the water not suitable, but still may be tolerated in the absence or an alternative source but up to the limits indicated under 'permissible' limit in the absence or alternate source' in col. 4, above which the sources will have to be rejected.

Table 7.7: Parameters Concerning Radioactive Substances

S.No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to Part of IS 14194	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
i)	Radioactive materials:				
	a) Alpha emitters Bq/L, Max	0.1	No relaxation	Part 2	-
	b) Beta emitters Bq/L, Max	1.0	No relaxation	Part 1	-

NOTE - It is recommended that the acceptable limit is to be implemented. Values in excess or those mentioned under 'acceptable' render the water not suitable, but still may be tolerated in the absence of an alternative source but up to the limits indicated under permissible limit in the absence or alternate source' in col. 4 above which the sources will have to be rejected.

Table 7.8: Pesticide Residues Limits and Test Method

S. No.	Pesticide	Limit µg/L	Method of test, Ref to	
			USEPA	AOAC/ ISO
(1)	*(2)*	*(3)*	*(4)*	*(5)*
i)	Alachlor	20	525.2, 507	-
ii)	Atrazine	2	525.2. 8141 A	-
iii)	Aldrin/Dieldrin	0.03	508	-
iv)	Alpha HCH	0.01	508	-
v)	Beta HCH	0.04	508	-
vi)	Butachlor	125	525.2, 8141 A	-
vii)	Chlorpyriphos	30	525.2. 814 1 A	-
viii)	Delta HCH	0.04	508	-
ix)	2.4-Dichlorophenoxyacetic acid	30	515.1	-
x)	DDT (*o, p* and *p, p* - Isomers of DDT, DDE and DDD)	1	508	AOAC 990.06
xi)	Endosulfan (alpha, beta, and sulphate)	0.4	508	AOAC 990.06
xii)	Ethion	3	1657 A	-
xiii)	Gamma -HCH (Lindane)	2	508	AOAC 990.06
xiv)	Isoproturon	9	532	-
xv)	Malathion	190	8141 A	-
xvi)	Methyl parathion	0.3	8141 A	ISO 10695
xvii)	Monocrotophos	1	8141 A	-
xviii)	Phorate	2	8141 A	-

NOTE - Test methods are for guidance and reference for testing laboratory. In case of two methods. USEPA method shall be the reference method.

Table 7.9: Bacteriological Quality of Drinking Water[1)]

S. No.	Organisms	Requirements
(1)	**(2)**	**(3)**
i)	All water intended for drinking: a) E. coli or thermotolerant coliform bacteria[2), 3)]	 Shall not be detectable in any 100 ml sample
ii)	Treated water entering the distribution system: a) E. coli or thermotolerant coliform bacteria[2)] b) Total coliform bacteria	 Shall not be detectable in any 100 ml sample Shall not be detectable in any 100 ml sample
iii)	Treated water in the distribution system: a) E. coli or thermotolerant bacteria b) Total coliform bacteria	 Shall not be detectable in any 100 ml sample Shall not be detectable in any 100 ml sample

1) Immediate investigative action shall be taken if either E. coli or total coliform bacteria are detected. The minimum action in the case of total coliform bacteria is repeat sampling; if

these bacteria are detected in the repeat sample, the cause shall be determined by immediate further investigation.

2) Although E. coli is the more precise indicator of faecal pollution, the count of thermotolerant coliform bacteria is an acceptable alternative. If necessary, proper confirmatory tests shall be carried out. Total coliform bacteria are not acceptable indicators of the sanitary quality of rural water supplies, particularly in tropical areas where many bacteria or no sanitary significance occur in almost all untreated supplies.
3) It is recognised that, in the great majority of rural water supplies in developing countries, faecal contamination is widespread. Under these conditions, the national surveillance agency should set medium term targets for progressive improvement of water supplies.

7.5 Water Quality Data

The water quality data of surface raw water source and ground water source are very crucial for designing and implementing the water supply systems. Thus, the water quality data should be collected from the government agencies such as Central Pollution Control Board (CPCB), Central Water Commission, and State Pollution Control Board, so as to design the treatment system. The available data with these agencies will cater to all quality fluctuations and shocks during the design period as these agencies collect data in various seasons. However, it is always a good practice to collect samples from the proposed water supply source as detailed in subsequent section. Hence, samples should also be collected from the prospective water source in addition to referring the data available with these agencies.

7.5.1 Surface Water Quality Data

In 1978, the Central Pollution Control Board began monitoring national water quality as part of the Global Environmental Monitoring System (GEMS) Water Programme. With 24 surface water and 11 groundwater stations, the monitoring programme was off to a start. In addition to GEMS, in 1984, a system for Surface Water, viz., a National Programme of Monitoring of Indian National Aquatic Resources (MINARS) was launched, with 113 stations spread across ten river basins. This monitoring network has been further expanded and many surface water sources including major lakes/ponds.

The monitoring activities carried out as part of the national network serves various assessment objectives as given below in determining:

- Natural freshwater characteristics without direct human influence,
- Long-term trends in critical water quality indicators in freshwater resources,
- Organic matter, suspended solids, nutrients, toxic chemicals, and other pollutants that flow from river systems to the sea/coastal interfaces.

A selective network of strategically vital monitoring stations has been established to achieve the earlier objectives. It is being operated in the country's major, medium, and minor watersheds of rivers, lakes, ponds and storage tanks, bodies of water, drains, water channels, and subsoil aquifers. Three types of stations are set up for monitoring: baseline, trend and flux stations. Periodic water quality monitoring is being undertaken by CPCB with assistance from State Pollution Control Boards. The water quality data of surface water sources are available on the website of CPCB.

7.5.2 Ground Water Quality Monitoring (GWQM)

GWQM is a co-ordinated effort to collect and analyse data on groundwater's physical, chemical, and biological characteristics, aquifer conditions, and designated ground use. Groundwater is characterised by contrasting aquifer and geologic features, limited accessibility (i.e., groundwater must be sampled through an existing or newly drilled well or spring), and 3-D distribution movement within a geologic framework. Central Ground Water Board (CGWB) co-ordinates GWQM through a network of observation wells spread across the country. Water quality along with water level data of these sources (observation wells) are periodically published in the form of reports by CGWB.

If any new water source/s is identified for water supply schemes and water quality data are not available for these sources, detailed analysis should be undertaken by analysing the parameters as specified in Table 8.1: Frequencies and parameters for analysis of surface water and groundwater sources (Part B of the Manual).

7.5.3 Water Quality Assessment

Assessment is the systematic documentation and study of the quality phenomenon to take decisions in designing the water supply system components, mainly intake and treatment processes. The assessment should take into account the following parameters:

(a) Where to sample: Locating the sampling point;

(b) What to measure: Define the parameters to be monitored;

(c) When to observe (how often): Frequency of observation.

After commissioning of the water supply system, it is vital to monitor the water quality parameters to ensure the proper functioning of the system. Water quality data collected during the water supply system operation should help to take any corrective changes required to deliver ‘Drink from Tap’ with 24×7 pressurised water supply. Refer Chapter 8 in part B of this Manual for detailed deliberation on monitoring aspects.

7.5.4 Critical Water Quality Assessment/Assurance Points

The quality of water being supplied through the water supply system has to be monitored to ensure conformance with the desired drinking water standard. The facility has to be in place to carry out these water quality tests at the critical points and transmit the data to the central unit to make positive changes in the design of various components to optimise the system.

These critical points must be carefully selected to capture variations in water quality in the water supply system. It is recommended to provide the following critical points to assess the water quality:

i) At the intake/tube well (water source);

ii) At the inlet and outlet of the treatment plant;

iii) At the inlet and outlet of service reservoirs such as ESRs/MBRs/GSRs;

iv) Inlet and at the farthest point in each District Metered Area (DMA);

v) At discrete locations in the network/DMA.

The emerging technologies, e.g., IoT and related instrumentation, can be used for efficient water quality monitoring at few of these critical points. Water quality data being collected by using emerging technologies should be connected to SCADA and analysed with digital twin technology for taking decisions and implementing corrective measures. The water quality data should be mapped on GIS.

Case Study

Digital Monitoring of 24×7 Water Supply (Drink from Tap Mission) in Puri and other Cities of Odisha

In the Indian context, intermittent water supply systems have several issues, including inefficient and poor design, concerns with maintenance and operation, financial instability, etc. Throughout the decades, urban Odisha was under great difficulty in terms of water supply infrastructure such as inadequate service coverage, poor drinking water quality, discontinuous supply and high-water losses, and could not keep up with the rate of expanding urbanisation. To tackle this problem, Government of Odisha initiated 'Drink from Tap' Mission, during October 2020 to provide good quality piped drinking water to approximately 2.5 lakh consumers of Puri city on a 24×7 basis. The objective of the mission was to make available sufficient quantity and good quality water directly from the tap. Water supply management was achieved through a community partnership known as "Jalsathi", which provided equitable, sustainable, and people-centred service provisions. It also focused on 100% coverage of both household connections and metering to avoid non-revenue water (NRW) and quality assurance through third-party quality monitoring and laboratories. Jal Sathis contributed to a change in the situation on the ground and increased public trust in the water supply system. The "Smart Water Management System" under "Drink from Tap Mission" was created and had initially been implemented in four pilot zones of Puri City to enable real-time data collecting, analysis, decision making, and public reporting (Figure A). Now it has been upscaled for pan city for Puri and additional 7 cities in Odisha.

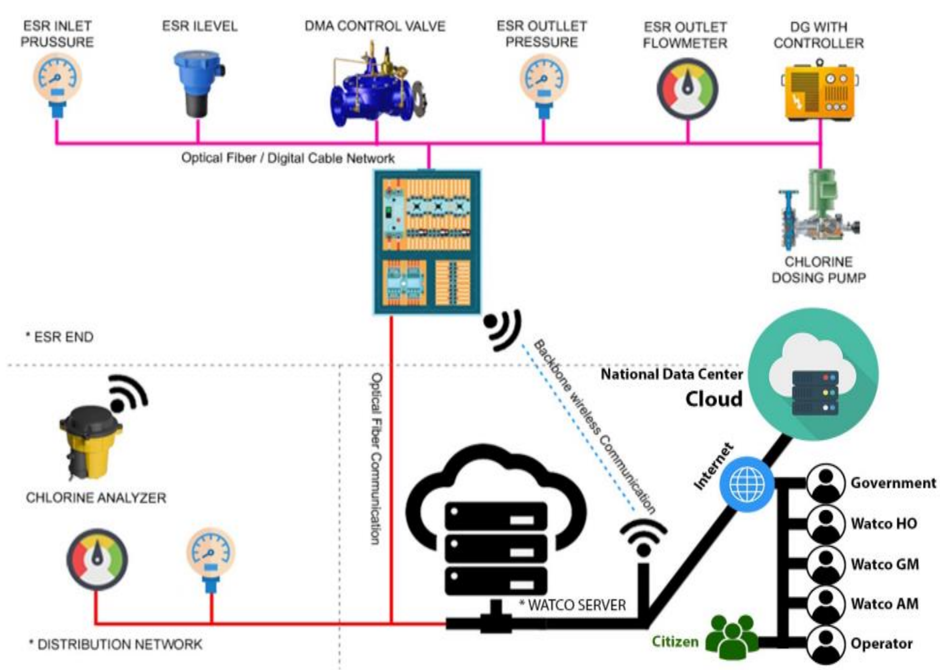


Figure A: System Architecture of Smart Water Management System in Puri

It aims to build public trust in the public water supply system so that individuals can drink from the tap instead of relying on the current delivery systems. In order to increase customer confidence, real-time online water quality data is broadcast on LCD panels in public areas (Figure B). For better water quality monitoring of essential parameters and effective incident management, mobile van laboratories have been deployed. The main difficulty has been reducing non-revenue water usage. To reduce leakage, a special cell with a committed crew has been developed, with implementation targeted at NRW reduction. The NRW is now less than 15%, down from a high of 54%. Special

Mobile Team have been established for speedy reaction to water supply-related incidents and immediate maintenance of leaks.

Figure B: Digital display board mounted at public places

The public and communities now have more faith in government because each home has access to high-quality water around-the-clock (24/7), there is no need for household storage, pumping, or treatment, house connections are simple, water is conserved through metering, production costs are reduced, issues and complaints are resolved quickly, and above all, end-to-end services are provided right at the doorsteps.

7.6 Establishing Testing Mechanism

Several institutions are catering to water testing requirements. An institutional mechanism of laboratories is proposed below that can be made functional at different levels, e.g., at National, State, and ULB level. Water quality laboratories are also the backbone of water quality monitoring and surveillance programme. Well-located and well-equipped analytical laboratories with competent staff are essential to evaluate water utility services' efficiency in terms of water quality. Therefore, the provision of safe drinking water warrants a strong laboratory network within the state and ULB for water quality assessment.

7.6.1 Proposed institutional mechanism of laboratories

As explained earlier, an institutional mechanism exists for water quality monitoring and surveillance at various levels. To synchronise water quality monitoring and surveillance and have a similar template for the data collection, compilation and analysis, existing mechanisms need strengthening.

Strengthening of the network of water quality assessment laboratories is based on a state laboratory and a series of laboratories at the ULB, including mobile and basic laboratories at the water treatment plant level (Figure 7.1). The laboratory at the water treatment plant can also serve the entire water supply system, including the function of the basic laboratory. ULB can have any of the following laboratory systems:

i) Two laboratories separately catering to (a) the source and distribution system and (b) the basic laboratory at water treatment plant;

ii) Single laboratory catering to the entire water supply system including water treatment plant.

In addition to ULB laboratories, a state laboratory is also proposed which may co-ordinate activities of all the ULB laboratories in the state. The state laboratory should be accredited by National Accreditation Board for Testing and Calibration Laboratories (NABL). The laboratory should be well-equipped to deal with the parameters identified in the Bureau of Indian Standards on quality standards for drinking water (IS 10500:2012). It is recommended that ULB laboratories should be subsequently accredited by NABL. It is required to perform external control on the quality of the analysis performed by ULB laboratories. State and large ULBs should also have mobile laboratories.

The ULB-level laboratories should be capable of carrying out a moderate series of physical, chemical, and microbiological analysis, which must be subject to quality assurance programmes to guarantee their quality. In addition, they should be able to offer support services to the field staff carrying out tests using portable equipment.

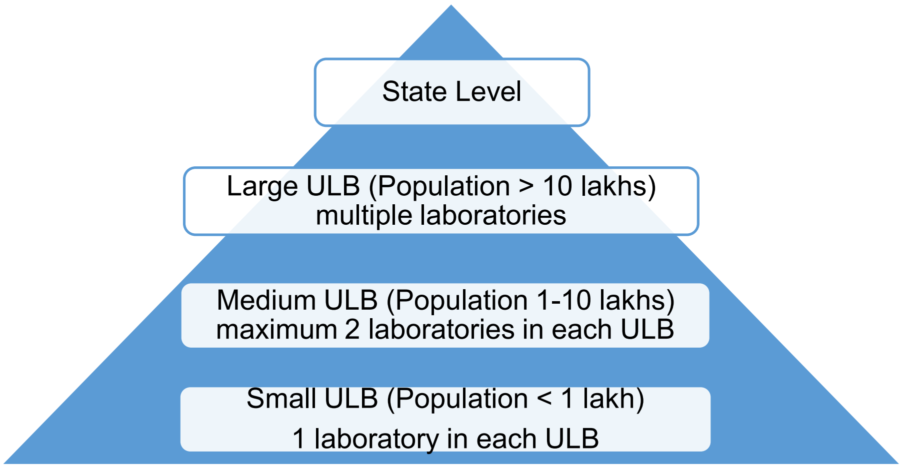


Figure 7.1: Laboratory Network for Water Quality Assessment

A minimum of one water testing laboratory should be established in each ULB. The State Laboratory of the concerned State Government Department (e.g., Urban Development Department) can co-ordinate with any of the laboratories being operated by other State and Central Government agencies (Public Health Engineering Department (PHED), Central Ground Water Board (CGWB), Central Water Commission (CWC), etc.) to optimise available resources.

Following guidelines can be adopted while locating water testing laboratories:

i. Only one water testing laboratory should be established in Small ULBs (Population <1 lakhs). Wastewater analysis should be undertaken in the same laboratory, however, proper precautions (such as microbiological analysis in separate rooms for water and wastewater analysis) should be undertaken to avoid cross-contamination. Similarly, specific water testing parameters such as Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), and Total Kjeldahl Nitrogen (TKN), etc., should preferably be analysed in a separate laboratory room.

ii. Maximum of two water testing laboratories can be established in Medium ULB (Population 1-10 lakhs).

iii. Large ULBs (>10 lakhs population) can have multiple laboratories, however, attempts should be made to minimise number of laboratories.

iv. Water testing laboratories should be in government-owned buildings.

v. It is a practice to locate water testing laboratory in the premises of water treatment plants. While the practice can be continued in locating the laboratory in water treatment plant premises, water testing laboratory can also be located in prominent location in the city/urban area to have better accessibility to the citizens. Establishing a water testing laboratory in the premises of Incharge of water supply in ULB will also help in providing prominence to these laboratories and also assist in better co-ordination between engineers and analysts/chemists.

vi. Location of water testing laboratory should be prominently displayed in busy public areas by using sign boards, wall paintings, etc., so that the community in general can be aware of laboratory facilities.

vii. Location of water testing laboratories should also be geo-tagged and displayed in the ULB website.

viii. It is a common practice to locate water testing and wastewater (sewage) testing laboratories in the premises of water treatment plant and sewage treatment plant respectively. This helps in optimising time for collection and analysis of water and wastewater samples from the treatment plants. While this practice can be continued if separate laboratories are already existing in water treatment and wastewater treatment plant premises.

ix. Joint or separate laboratories for water and wastewater analysis can be planned and operationalised. If it is proposed to have water testing and wastewater testing laboratories in the same premises/buildings, microbiology sections for water and wastewater analysis should be separate to avoid cross-contamination. Similarly, specific water testing parameters such as Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), and Total Kjeldahl Nitrogen (TKN), should preferably be analysed in a separate laboratory room.

x. Staff for wastewater analysis should be separate and same guidelines available for water quality analysis can be adopted for engagement of staff.

xi. Major equipment such as Atomic Absorption Spectrophotometer (AAS) with Electrode Lamp, Inductively Coupled Plasma - Optical Emission Spectrometry (ICP-OES)/Inductively Coupled Plasma - Mass Spectrometry (ICP-MS), Gas Chromatograph - Mass Spectrometry (GC-MS), Liquid Chromatograph - Mass Spectrometry (LC-MS), Ion Chromatograph (IC) and PCR machine should be shared between large laboratories for water and wastewater analysis. Only one set of these equipment should be procured in large ULB (population >10 lakhs) and in the State Laboratory even if there are multiple water testing laboratories in any ULB.

xii. State laboratory should preferably be located in the capital city of the State.

7.6.2 Functions of Water Quality Testing Laboratories

Within the proposed hierarchy, the basic laboratory at WTP controls and optimises the treatment process. In case of a plant of greater capacity (>50 MLD), all the additional parameters shall be monitored in the ULB laboratory.

In general, the critical function of a water testing laboratory is to determine the water quality for drinking and domestic use. To undertake this function, the following activities are involved, viz.:

(i) Collection of water samples from the field with suitable preservation

(ii) Sanitary surveillance

(iii) Water sample storage with suitable preservation

(iv) Requisite data analysis

(v) The other functions would include:
- Delineating the potential areas of water contamination (health-based parameters like excess of salinity, fluoride and metals such as iron, manganese, arsenic, etc.);

- Determining the risk of pollution from various sources like agricultural practices (pesticides and fertilisers), industrial discharges, municipal sewage disposal and disposal of solid wastes;
- Communicating the results to concerned officials for corrective actions;
- Follow-up water quality monitoring after implementation of corrective actions, mainly if the source is bacteriologically contaminated;
- Identifying sampling stations and frequency; and
- Providing reference/critical points to monitor improvement or deterioration in water quality.

The concerned authorities in ULBs will carry out the following activities:

i.) Set up/strengthen ULB-level laboratories.

ii.) Upgrade existing water quality testing laboratories, including procurement of equipment, instruments, chemicals/reagents, glassware, consumables, etc.

iii.) Procure special vehicles for transportation of water samples from the field to the laboratory.

iv.) Conduct Information, Education and Communication (IEC) activities on the importance of consuming safe drinking water, capacity building, and training various stakeholders.

v.) Seek accreditation for ULB-level laboratories through the NABL.

The envisaged functions of different laboratories within the proposed hierarchy and requisite staffing shall be as given below in Table 7.10.

Table 7.10: Envisaged Functions of Various Laboratories

Laboratory	Envisaged Functions
State level Laboratory and large ULB (10 lakhs or greater)	It will be a well-equipped laboratory capable of: • Analysing the physico-chemical and microbiological parameters of water and wastewater • Supervising and assisting medium and small laboratories in the state on sampling, water quality analysis, data analysis, and crosschecking of standards • Preparing and implementing water safety plan • Undertaking routine monitoring and surveillance of the water supply system and suggest corrective actions based on water quality analysis data • Routine monitoring of identified control measures within the water supply system • Identifying contamination points within water supply systems and control • Validating and entering data in a standard database • Undertaking analysis of all routine water quality parameters, viz., heavy metals, pesticides, bacteriological, and biological analysis with sophisticated instruments as detailed in Chapter 8 of Part B Manual • Taking up independent public health surveillance for water safety • Carrying out community awareness programmes • Collaborating with similar water testing laboratories in the State, e.g., State

Laboratory	Envisaged Functions
	Laboratory of Rural Water Supply Agency (PHED)
Medium (1-10 lakhs) and small ULB (<1 lakh) laboratory, and basic laboratory at WTP	It will be a well-equipped laboratory capable to: • Analyse specified required water quality parameters, as mentioned in Chapter 8 - Part B Manual • Undertake routine monitoring and surveillance of the water supply system • Prepare and implement water safety plan • Enter data in a standard database • Carry out community awareness programmes • Collaborate with similar water testing laboratories in the State, e.g. District Laboratory of Rural Water Supply Agency (PHED)

7.6.3 Mobile Drinking Water Quality Testing Laboratory

ULBs can procure mobile water quality testing laboratories to test specified endemic parameters, local communities' congregation areas, and inaccessible parts of the city. This type of mobile laboratory may be needed in areas where water samples cannot be transported to designated laboratories on time. The mobile laboratories should be fully equipped to conduct on-the-spot water analysis concerning drinking water sources' safety. The results of the tests will help determine strengthening of preventive measures, treatment needed and warning to consumers to make the contaminated water safe to drink. They play a significant role during calamities/disasters as they can reach these areas quickly, i.e., cyclone and flood-prone areas, landslides, and earthquake-prone areas, etc. The primary functions of a mobile laboratory include:

i. Water quality testing, monitoring and surveillance in remote areas/hot spots/disaster-prone areas;
ii. Cross-verification of results with other laboratories;
iii. Water quality testing and management during disasters and natural calamities;
iv. Awareness generation amongst the community.

7.6.4 Staffing

Staff requirements for water quality testing varies widely according to the population of ULB, number of water samples to be collected and analysed, number of water sources, treatment plant size, and financial resources. A suggestive staff requirement for various levels of laboratories is given below in Table 7.11 and Table 7.12. The requisite qualification of each staff can be based on the concerned State Government guidelines. The position of Chief Water Analyst should be held by chemist/microbiologist/environmental scientist having experience in water quality analysis and surveillance. The staff of the laboratories namely chemists, analysts, microbiologists, bacteriologists, and laboratory assistants should have expertise and experience, and must understand the principle and operate the equipment/instruments available in the laboratory.

Table 7.11: State/ULB-Level Water Quality Testing Laboratory

S. No.	Position	Numbers			
		State Laboratory	ULB		
			Large ULB (population >10 lakhs)	Medium ULB (1-10 lakhs)	Small ULB (<1 lakh)
1.	Chief Chemist/Water Analyst	01	01	01	-
2.	Senior Chemist/Water Analyst	02	01	01	-
3.	Environmental Engineer	02	01	01	-
4.	Chemist/Water Analyst	03	02	01	01
5.	Microbiologist/Bacteriologist	02	02	01	01
6.	Laboratory Assistant	04	03	02	01
7.	Lab Attendant	06	04	03	02
8.	Data Entry Operator	02	02	01	01
9.	Field Assistant/sample collector (task/need-based field staff)	06	04	03	02

Table 7.12: Mobile Water Testing Laboratory

S. No.	Position	Numbers
1.	Chemist/Water Analyst/Microbiologist	01
2.	Field Assistant (task/need-based field staff)	01
3.	Driver	01
4.	Helper	01

7.7 Laboratory Facilities and Equipment

7.7.1 Facilities

At the time of construction of water works assets, it is critical to provide testing laboratories and procure applicable standard instruments and equipment. The laboratory's layout will be determined by the various analytical work that must be completed. When choosing the required space, consideration should be given to the space needed for permanently installed equipment and the efficient completion of analytical work by laboratory workers. Future growth should be considered while building the new laboratory or retrofitting the old laboratory. Urban areas for developing separate structures of laboratory facilities are categorised as follows:

i) Large ULB (Metropolitan areas having a population >10 lakhs).

ii) Medium ULB (Municipal Corporations having a population 1-10 lakhs).

iii) Small ULB (Municipalities having a population <1 lakh).

Although these laboratories will be operated and maintained by individual ULBs, cross-linkages among laboratories should be encouraged in a State for optimising resource availability. Table 7.13 lists the necessary laboratory equipment.

Table 7.13: Facilities Required in a Laboratory

S. No.	Infrastructure	State or Large ULB Laboratory (population > 10 lakhs)	Medium ULB Laboratory (population 1-10 lakhs)	Small ULB Laboratory (population <1 lakh)
1.	Space for Analysis (minimum area)	80 m^2 (including 20 m^2 biological)	60 m^2 (including 20 m^2 for biological testing)	50 m^2 (including 10 m^2 for biological testing)
	Space for storage (in m^2)	45	25	20
	Space for office & library (in m^2)	45	15	10
	Total space req. (in m^2)	170	100	80
2.	No. of Computers	03 (include one system for library)	01	01
3.	Internet facility	Yes	Yes	Yes
4.	No. of UPS	02	01	01
5.	Inverters (back up time - 3 hrs.)	02	02	01
6.	Printer	02	01	01
7.	Telephone Facility	Yes	Yes	Yes
8.	Fax	Yes	Yes	Yes
9.	A.C.	Yes	Yes	Yes
10.	Provision for Fume hood	Yes	Yes	May not be needed at this level
11.	Biosafety cabinet	Yes	Yes	Yes
12.	Provision for gas connection	Yes	Yes	Yes (Only LPG)

Additional space for wastewater treatment/ spent water generated from the laboratory should be provided.

(Source: Drinking Water Quality Monitoring & Surveillance Framework, 2021 of GOI)

7.7.2 Equipment

Water quality testing equipment are staple in environmental laboratories. Various types of water quality testing equipment are used to test water for physical, chemical, and microbiological contaminants. These equipment can be used to test a variety of parameters in water. Equipment to be procured by municipal corporations and other ULBs should be decided by the concerned ULB based on water quality parameters to be analysed. It is recommended that the state and large ULBs should have facilities to analyse all the water quality parameters, as mentioned in Table 7.14. The laboratory should cater to other ULBs (particularly medium and small ULBs) in case episodic monitoring needs to be carried out and other laboratories don't have facilities to analyse the concerned water quality parameters. The equipment listing is based on the incremental approach for choosing the parameters to be tested at each level of laboratories. Each ULB should ensure that an appropriate laboratory infrastructure as recommended by NABL should be first created before procuring the equipment. Moreover, qualified and experienced/trained staff availability should also be ensured before procurement of the equipment. However, non-availability of trained staff should be an excuse to avoid carrying out water quality analysis. The water quality analysis of the listed parameters is essential and services of any other NABL accredited laboratories as detailed out in Section 8.5 (Part B Manual - Chapter 8) should be utilised.

Table 7.14: Equipment/Instrument in water testing laboratories (Indicative numbers given against each equipment)

S. No.	Item	State	Large ULB	Medium ULB	Small ULB
1.	pH meter (each laboratory based and portable type)	2	2	1	1
2.	TDS/Conductivity meter (each laboratory based and potable type)	2	2	1	1
3.	Nephelometer (Turbidimeter) (each laboratory based and potable type)	2	2	1	1
4.	Digital balance	2	2	1	1
5.	UV-Visible spectrophotometer	2	2	1	1
6.	Refrigerator	2	2	1	1
7.	Water still	2	2	1	1
8.	Voltage stabiliser/Inverters	3	3	2	2
9.	Hot Plate	2	2	1	1
10.	Heating mantle	2	2	1	1
11.	Water bath	2	2	1	1
12.	Hot air oven	4	4	2	2
13.	Bacterial incubator	3	3	2	1
14.	Membrane filtration unit	2	2	1	1
15.	Autoclave	2	2	1	1
16.	Magnetic stirrer	2	2	1	1
17.	Microscope	1	1	1	1
18.	UV laminar air flow chamber for bacteriological analysis	1	1	1	1
19.	Plate count and colony counter	1	1	1	1
20.	Arsenic testing instrumentation (portable) - only in affected states	1	1	1	1
21.	Hydride generator with all accessories	Yes	Yes	Yes	Yes
22.	DO meter	1	1	1	1

S. No.	Item	State	Large ULB	Medium ULB	Small ULB
23.	Specific ion meter along with electrodes (for fluoride)	1	1	1	1
24.	Fume cupboard	2	2	1	1
25.	Auto burette and auto pipette (numbers can be decided as per requirements)	Yes	Yes	Yes	Yes
26.	Thermometers (numbers can be decided as per requirements)	Yes	Yes	Yes	Yes
27.	Double distillation apparatus/Ultrapure water purification system to provide type I/type II water for sophisticated instruments	2	2	1	1
28.	Argon, Nitrogen, Hydrogen Helium and Oxygen Gas Cylinders (for AAS/Advanced Spectro-photometer) / ICP-MS/OES	Yes	Yes	Yes	No
29.	Kjeldal distillation apparatus	Yes	Yes	Yes	No
30.	Pressure pump	Yes	Yes	Yes	No
31.	Deep freezer (-20 °C)	1	1	No	No
32.	Micropipette	Yes	Yes	No	No
33.	Centrifuge	2	2	1	1
34.	Reflux apparatus/COD digester	1	1	1	No
35.	Ion chromatograph	1	1	No	No
36.	Atomic absorption spectrophotometer (AAS) with electrode lamp	1	1	No	No
37.	Inductively coupled plasma- optical emission spectrometry (ICP-OES)/mass spectrometry (MS)	1	1	No	No
38.	Gas chromatograph-mass spectrometry (GC-MS)	1	1	No	No
39.	Liquid chromatograph-mass spectrometry (LC-MS)	1	1	No	No
40.	PCR machine	1	1	No	No

Required laboratory consumables (glassware and accessories) and chemicals in the required quantity should also be maintained in stores of each laboratory.

7.8 Water Quality Index (WQI)

A water quality index provides a single number (like a grade) that expresses the overall water quality of a particular water sample (location and time specific) for several water quality parameters. Developing an index aims to simplify the complex water quality parametric data into comprehensive information for easy understanding. A water quality index based on important parameters provides a simple indicator of water quality which gives a general idea of the possible problems with the water in the region and across the stretch of the river/stream, helping in deciding the best alternative location of intake. Hence, water quality index should be determined water sources to be used for water supply. Water quality parameters, namely, dissolved oxygen, faecal coliforms, pH and BOD (as mentioned in Table 7.15) should be used to determine water quality index.

The WQI has been determined based on the formula developed by National Sanitation Foundation (NSF) and modified by Central Pollution Control Board (CPCB), which depicts the water quality simply and easily for utilities and the general public at large. To maintain uniformity while comparing the WQI across the nation, the NSF-developed WQI has been modified, and CPCB has assigned relative weights. The modified weights are given in Table 7.15. The formula and classification of water quality indices for surface and groundwater are given in Table 7.16. The water quality is described upon determining the water quality index for easy understanding and interpretation.

Table 7.15: Modified Weights for Computation of WQI based on DO, FC, pH and BOD

Parameters	Original Weights from NSF WQI	Modified Weights by CPCB
Dissolved oxygen (DO)	0.17	0.31
Faecal coliform (FC)	0.15	0.28
pH	0.12	0.22
BOD	0.1	0.19
Total	0.54	1

Table 7.16: Formula and Classification of Water quality indices for Surface and Groundwater

Surface Water Quality	Ground Water Quality
$WQI = \sum_{i=1}^{P} W_i I_i$	$WQI = \sum_{i=1}^{n=9} q_i . W_i$
Where: I_i = sub-index for water quality parameter, Where, sub-index = monitored concentration/water quality standard W_i = weight (in terms of importance) associated with water quality parameter P = number of water quality parameters	Where: q_i = quality rating, $= \frac{C_i}{S_i} \times 100$ W_i = the relative weight $= \frac{w_i}{\sum_{i=1}^{n} w_i}$ w_i = the weight of each parameter or relative of each weight C_i = the concentration of each chemical parameter in each water sample in mg/L S_i = the Indian drinking water standard for each chemical parameter in mg/L according to the guidelines of the BIS 10500, (2012)

WQI	Quality classification	Remarks	Colour code
Surface Water Quality			
63-100	Good to Excellent	Non-polluted	
50-63	Medium to Good	Non-polluted	

38-50	Bad	Polluted	
38 and less	Bad to Very Bad	Heavily polluted	
Ground Water Quality			
<50	Excellent	Non-polluted	
50-100	Good Water	Non-polluted	
100-200	Poor Water	Polluted	
200-300	Very Poor	Polluted	
>300	Water unsuitable for drinking	Heavily polluted	

7.8.1 Advantages of WQI

The following are the advantages of WQI:

- Reduces the number of parameters needed to compare water quality for a specific application.
- Provides a single number that represents overall water quality to a particular location and time.
- Identifies water quality in terms of time and space dynamics.
- Assures water body's safety to users, such as aquatic life habitat and drinking water supplies.
- Serves as a great way to monitor water quality.
- Allows comparisons between various rivers and sampling sites.
- Simplifies complex dataset into information that is simple to comprehend and use.
- Provides a single-value output derived from several parameters as well as important information about water quality that the general public and non-technical population can understand.
- Serves as a valuable tool for disseminating information about water quality to the general public and legislative decision-makers.

7.8.2 Limitations of WQI

Despite the many benefits of the WQI, it is beset by specific challenges.

- The WQI is not an absolute standard of pollution or water quality.
- There is a lack of precision and accuracy in the classification method of the significance of parameter evaluation.
- It is ineffective in mitigating risk and subjective experience in a complex environmental problem, such as observation incompatibility, uncertainty, and criteria imprecision.
- There is a lack of a standardised method for measuring biological parameters in water pollution.
- The transfer of critical environmental data into meaningful information is insufficient.

7.9 Sanitary Surveillance

The sanitary surveillance (survey) should include the location of all potential and existing health hazards. The information obtained from a sanitary survey is essential for evaluating the microbiological and chemical water quality data. It is desirable to:

i) Identify potential hazards;

ii) Determine factors that affect water quality.

The sanitary survey is elaborately discussed in section 8.7 in Part B of this manual.

The following are some factors that should be investigated in a sanitary survey.

7.9.1 Surface Water

i. Proximity to watershed and character of sources of contamination, including industrial wastes, oil field brines, acid waters from mines, sanitary landfills and agricultural drain waters.

ii. Population and wastewater collection, treatment, and disposal in the watershed.

iii. Closeness of sources of faecal pollution to intake of water supply.

iv. Wind direction and velocity data, the drift of pollution, and algae/aquatic growth potential in case of lake or reservoir supplies.

v. Character and quality of raw water.

vi. Protective measures in the watershed to control fishing, boating, car washing, swimming, wading, ice cutting and permitting animals on shoreline areas.

vii. Efficiency and constancy of surveillance on the watershed and around the surface water source.

7.9.2 Ground Water

i. Nature, distance, and direction of local sources of pollution including pit latrines, twin pits, etc.

ii. Possibility of surface-drainage water entering the supply and of wells becoming flooded.

iii. Drawdown when pumps are in operation, recovery rate when pumps are off.

iv. Methods for protecting the water supply against contamination from wastewater collection, treatment facilities, and waste disposal sites.

v. The presence of an unsafe supply nearby and the possibility of cross-connections cause a danger to public health.

vi. Occurrence of geogenic contaminants such as arsenic, fluoride, etc.

vii. Disinfection: equipment, supervision, test kits, or other types of laboratory control.

7.10 Water Safety Plan (WSP)

"The most effective means of consistently ensuring the safety of a drinking water supply is through the use of a comprehensive risk assessment and risk management approach that encompasses all steps in water supply from catchment to consumer." In the Guidelines for Drinking Water Quality, Fourth Edition, published by the World Health Organisation (WHO) in 2011, such an approach is termed a 'Water Safety Plan' (WSP).

The purpose of a WSP is to consistently ensure the safety and acceptability of a drinking water supply by adopting principles of preventive management. This is done by eliminating/minimising potential sources of contamination in the catchment, raw water sources, water treatment plants, distribution network, storage, collection, and handling. The concept was drawn from traditional multiple-barrier risk management techniques and the Hazard Analysis at Critical Control Point (HACCP) approach, which has been applied in the food manufacturing industry for several decades. This forms the basis of the development of the WSP approach. WSP can be developed for and applied to a large, piped drinking water supplies, small community supplies and household systems.

Over the last decade, WSPs have gained acceptance as an essential framework for achieving water quality and health-based targets. Public water utilities in Australia, the United Kingdom, Latin America and the Caribbean, Bangladesh and Uganda have successfully developed and implemented WSPs for their water supply systems. Case studies document increased compliance with drinking water quality regulations, improved watershed management practices, reduced cost of operation and significant cost savings as a result of implementing WSPs.

A practical risk management approach is required to resolve the issues of urban water, particularly with respect to inferior water quality. WSP will help in implementing 'Drink from Tap' initiative.

7.10.1 Preparation and Implementation of Water Safety Plan

There are five key components to any WSP:

(i) *System assessment*, which determines if the drinking water supply system, from catchment to consumer, is capable of supplying water of sufficiently high standards to meet regulatory targets. A thorough understanding of the supply system is necessary before the WSP can be developed. For this, a multidisciplinary team familiar with the drinking water supply system should be formed to carry out risk assessment and hazard identification at each process step.

(ii) *Operational monitoring*, to define and validate the monitoring of water quality parameters at each process step and the control measures associated with them. This is done to identify any failures in the system as soon as they occur so that corrective actions can be taken without delay.

(iii) *Management plans*, which include supporting programmes, describe actions taken during normal and incident operational conditions and define a schedule for review of the WSP. This ensures the correct implementation and continued effectiveness of the WSP in meeting water quality standards.

(iv) Management and communication.

(v) WSP review and improvement.

It is recommended that ULBs should prepare and implement water safety plans. Preparing and implementing a water safety plan will help to ensure water safety in the water supply system and match International best practices. World Health Organisation (2023), Water safety plan manual: step-by-step risk management for drinking water suppliers, Second Edition should be used to prepare WSP:

There are 10 modules (Table 7.17) in the Manual which will help in preparing water safety plan document by ULB.

Table 7.17: Water Safety Planning: Overview of the 10 Modules

WSP Component	Module No.	Title	Addresses
Preparation	1	Assembling the WSP Team	Who will lead WSP development and implementation?
System Assessment	2	Describing the System	How does the system deliver drinking water from catchment to consumer?

WSP Component	Module No.	Title	Addresses
	3	Identifying hazards and hazardous events	What could go wrong?
	4	Validating existing control measures and assessing risks	How effective are the control measures and how important are the risks?
	5	Planning for improvement	What needs to be improved to ensure the supply of safe drinking water, and how?
Monitoring	6	Monitoring control measures	Are the control measures operating as intended?
	7	Verifying the effectiveness of water safety planning	How do we know that the WSP is working and effective?
Management and Communication	8	Strengthening management procedures	What management procedures should be used for normal and abnormal conditions?
	9	Strengthening WSP supporting programmes	What is the best way to support the implementation of water safety planning?
WSP review and improvement	10	Reviewing and updating the WSP	How will the WSP be kept up to date?

Case studies from India are presented below which can be referred while preparing water safety plan:

Case Study

Hazard in the catchment and implementation of control measures

Figure 1: Ash Pond in the Catchment of Kanhan River

(Intake well of Kanhan WTP encircled, and change in colour of water due to overflow of breached ash pond in February 2017)

Kanhan river is a major source of water supply to Nagpur, India. A water safety plan was prepared in 2011 and updated in 2014. The ash pond of Koradi/Khaparkheda thermal power plants upstream of the intake point in the catchment was identified as a major hazard. Supernatant/overflow from the ash pond to the intake point in Kanhan River was identified as a major hazardous event in WSP-2011. In contrast, the breach is identified as another hazardous event in WSP-2014 (this was missed in earlier WSP). Although existing control measures, such as a bund around the ash pond, were in place, the residual risk was identified as high. Control measures like co-ordination among thermal power plant authorities, water suppliers and statutory authorities were also recommended. Another measure was a stoppage of the intake well pumping if there was the slightest evidence of ash in the river water. Water suppliers implemented these measures. There was an accidental breach in the ash pond in February 2017 (Figure 1), and because of the control measures in place, the water supply from Kanhan river could be normalised in a couple of days which otherwise could have taken weeks.

Case Study

Hazard Identification and implementation of control measures in Master Balancing Reservoir

The Master Balancing Reservoir (MBR), with a capacity of about 2.5 million litres located in Seminary Hills, was in a state of disrepair. Leakage was observed at its inlet valve (Figure 2), which was identified as a major hazard during the WSP preparation in 2011. The slabs of the inlet chamber were in need of repair and the iron reinforcement rods were corroded. Control measures included the replacement of the valve as the risk was very high. The water supplier implemented and replaced the valve (Figure 3).

Figure 2: Leaking valve identified as a very high risk during WSP preparation

Figure 3: Replacement of leaking valve (an example of control measure implementation)

CHAPTER 8: CONVENTIONAL WATER TREATMENT

8.1 Introduction

The objective of water treatment is to treat source water, in a cost-effective manner that meets drinking water standards of BIS code IS 10500 (2012) and its amendments.

8.1.1 Methods of Treatment

The method of treatment to be employed depends on the nature of raw water constituents. The traditional surface water sources are streams, rivers, tributaries, manmade and natural lakes, and canals. Traditional groundwater sources are dug wells and bore wells. The seawater, brackish water, and reuse of treated sewage (indirect potable use) are considered as alternative sources. The impurities in raw water can be classified as suspended solids, colloidal solids, and dissolved solids which may be harmful to humans. Impurities also can be classified as inorganic, organic, and microbial contaminants. For drinking water treatment to remove suspended or precipitated solids and contaminants, unit processes or their combination, viz., coagulation, flocculation, clarification, filtration, membrane, and disinfection is a must. The degree of treatment processes required is directly related to the quality of raw water. Recently, biological filtration (after conventional filtration) is also included to remove organics and Disinfection By-Products (DBPs) to some extent. Biological Filtration means conventional high rate filters backwashed without chlorinated water. The degree of treatment shall be directly related to the quality of raw water including traditional and emergent contaminants. Chapter 7 of the Part A Manual discusses various types of impurities that could be present in the raw water.

Sometimes, a highly polluted stretches of water due to discharge of wastewater (organic matter) are encountered, which require much higher chlorine doses, however, adding excessive chlorine is dangerous and leads to formations of DBPs. In such situation, different treatment technology such as enhanced coagulation, pressure or submersible membranes has to be resorted to (details are given in Part A chapter 10 Specific water treatment system).

Typical sizing of units of conventional WTP is enclosed in **Annexure 8.1**.

8.1.2 Desirable Raw Water Quality for Conventional Treatment

As mentioned earlier, coagulation, flocculation, clarification/precipitation, filtration and disinfection are the main unit operations and processes classified as conventional treatment. Normally, these processes and units are employed to treat surface waters and groundwater influenced by surface water. The surface water sources in India have predominantly inorganic suspended solids. However, in the lean flow season due to pollution by sewage, industrial wastewater, farm discharges organic suspended and colloidal impurities also occur in the raw water. Coloured water (due to natural decay of vegetation) also contributes to organic load. Microbial contaminants, viz., algae are also present in many polluted waters in lean flow season. Jar tests with different coagulants and coagulant aid give a representative analysis of settleable floc formation. BOD should not be ideally more than 3 mg/L as per Standards notified by CPCB in 2007 as mentioned in the table 7.3 on Chapter 7 of Part A of this manual. However, if the raw water source is having BOD of 5 mg/L or less, it is recommended to adopt conventional treatment with post chlorination.

8.1.3 Suggested Line of Treatment Options for Contaminated Surface Water Sources

Due to excessive river pollution as a result of constituents mentioned in the Chapter 7, the organic and microbial contaminants increase beyond the desirable limits. Conventional technologies become ineffective for BOD value consistently exceeding 5 mg/L throughout the year, therefore, the options of alternative/advanced technologies are needed to be adopted.

a. Status of Polluted River Stretches as per CPCB Report, 2022

As per the Report "Polluted River Stretches for Restoration of Water Quality" – 2022, Central Pollution Control Board (CPCB) assessed the water quality of 603 rivers of which 311 stretches of 279 rivers in 30 States and Union Territories are polluted (BOD more than 3 mg/L the Surface Water Quality Criteria of CPCB for drinking water sources). Following is the status of polluted river stretches in terms of BOD given in Table 8.1:

Table 8.1: Status of Polluted River Stretches in terms of BOD

Priority	Number of polluted stretches	No. of States/UTs	BOD Level mg/L
Priority 1	46	18	**More than 30**
Priority 2	16	10	**20 - 30**
Priority 3	39	16	**10 - 20**
Priority 4	65	21	**6 - 10**
Priority 5	145	26	**3 - 6**
Total	**311**	-	**-**

Progressively over the past, the wastewater (partially treated or untreated) has been discharged into the water bodies (traditional water sources). As the wastewater effluent and the pesticides leached from the agricultural fields discharged into the water bodies contain various contaminants including micro-pollutants, Endocrine Disrupting Chemicals etc., it is important to adopt high degree of treatment to protect the public health from the water related *life threatening* diseases. Conventionally, when organic content of source water is high, the pre-chlorination process in use may form Disinfection By-Products (DBPs) including Trihalomethanes (THMs), which are suspected carcinogenic. Therefore, the conventional water treatment processes are ineffective to treat such variety of contaminants. The BOD is an indicator of contamination of the raw water source as most of the river stretches in India have high BOD as stated above.

It has been observed that ULBs are making huge investments for treating wastewater where the cost of conventional wastewater treatment per MLD is 3 to 4 times than that of the conventional water treatment plant. In order to maintain drinking water quality as per BIS and supply safe and protected water with a view to safeguard public health, the degree of treatment for raw water sources may be required to be enhanced with a judicious combination of appropriate treatment technologies by making additional investments.

b. Nature of contaminants during lean flow season:

1. The contaminants during the lean flow season are a mixture of suspended and dissolved inorganic and organic solids.
2. These are difficult to treat in conventional treatment processes comprising aeration, coagulation, flocculation, clarification, filtration, and disinfection
3. Inorganic solids have a fast settling velocity, as against the smaller velocity of biological solids. It is a mix, depending on the raw water source.

c. Constitution of biological solids (Soluble or non-soluble)

1. NOM (Natural Organic Material – Humic and Fulvic acids) or SOM (Synthetic Organic Material) contribute to BOD, COD, and TOC in the raw water.
2. Algae is another pollutant that grows due to the presence of Nitrogen & Phosphorus contributed by untreated or partially treated sewage in the raw water.

While the following multi-prong strategy can be adopted by the State Government/ULBs to improve surface water source quality, treatment of low-strength remains inevitable at several locations:

1. Improving compliance with Hon'ble National Green Tribunal wastewater discharge standards by augmenting (wastewater) treatment plants
2. Catchment protection
3. Maintaining minimum environmental (ecological) flow in rivers for sufficient dilution

ULBs should appropriately identify drinking water sources by avoiding polluted stretches. The river stretches having the value of BOD_5 equal to or less than 5 mg/L can be directly used as a source for water supply.

Further, attempts by the State authorities/ULBs should be made to,

(i) improve source water quality by addition of a complementary treatment unit in the existing water treatment plant to treat polluted (alternatively low-strength wastewater) water in such river stretches, and
(ii) manage the ecological quality of rivers by maintaining a minimum flow (Rivers must not dry-up or have their physical regimes significantly altered in order to conserve the hydrological and ecological functions of their drainage networks) since, Hon'ble National Green Tribunal (NGT) in OA 498 of 2015 has directed all States to maintain a minimum environmental flow of 15-20% of the average lean season flow in their rivers.
(iii) The river authorities should consider the above flows (15-20%) to be discharged in the river to maintain the computation of water balance of the river sub basin.

d. Recommended Treatment Technologies for Source Raw Water with BOD

The BOD criteria for a raw surface water source to be used for drinking purpose as per CPCB is 3mg/L or less with conventional treatment. The BOD criteria may be relaxed up to 5mg/L with conventional treatment systems. However, the BOD level is varying from 3mg/L to more than 30mg/L in many Indian River water stretches. Thus, the following treatment technological Options/Units, are recommended in Table 1 for different range of BOD levels:

Table 8.2: Recommended Treatment Technological Options/Units for Raw Water with different BOD levels

Sr. No.	BOD_5 in mg/L	Recommended Technology/Units
1	**5 or less**	Conventional treatment with post chlorination to maintain the residual chlorine upto 0.2 mg/L at the fag end of the distribution system. If algae is found in raw water the necessary pre-treatment shall be given as suggested in Chapter 10.
2	**> 5 - 10**	Conventional treatment with Enhanced Coagulation - UV – post chlorination
3	**> 10 - 20**	Cascade aeration – conventional treatment with enhanced coagulation – Tertiary treatment such as Granular /*Biological Activated Carbon (GAV/BAC) – UV and chlorination

Sr. No.	BOD_5 in mg/L	Recommended Technology/Units
4	**>20 - 30**	**Option1**: Cascade aeration – conventional treatment with enhanced coagulation - tertiary treatment – Ozone, Granular /Biological Activated Carbon (GAV/BAC) – UV (optional as it reduces the chlorine dose) and minimal dose of chlorination to disinfect the residual micro-organisms in the distribution pipelines **Option 2**: Cascade aeration – Sedimentation (if turbidity > 50 NTU) - Granular /Biological AC with Empty Bed Contact Time (EBCT) 10 minutes, ultrafiltration– UV and chlorination to disinfect the residual micro-organisms in the distribution pipelines
5	**> 30 - 50**	Attached Growth Biological Treatment - ultrafiltration, Ultra Violet (UV) and minimal dose of chlorination to disinfect the residual micro-organisms in the distribution pipelines
6	**Raw water having BOD_5 > 50**	Attached Growth Biological Treatment - Granular Activated Carbon, Ultrafiltration, Ultra Violet (UV). This treated water is added to water bodies like lakes, ponds as an environmental buffer and then drawn into conventional water treatment for further treatment as desired with chlorination to disinfect the residual micro-organisms in the distribution pipelines. In case the TDS levels are significantly higher (more than 2000 mg/L) reverse osmosis (RO) can be used after ultrafiltration. This shall be considered in the absence of any alternative source.

Note:

i. It is advisable to carry out water quality analysis of the raw water sources and conduct pilot scale of 0.1 MLD studies/demonstration of the above stated options (S. Nos. 3, 4, 5 & 6) for raw water having BOD_5 more than 10 mg/L to ascertain design and operational parameter before embarking on large scale treatment plants. Public Awareness Programs / Campaigns shall be conducted before the project implementation.

ii. When the water quality changes w.r.t. time in the river or any water source, design the treatment for the worst scenario considering the water quality of the raw water sources during the last 5 years in consultation with the state board. Based on the quality required, treatment units (some of the units may require by-pass) may be operated.

iii. Various Water Treatment Systems/Processes/Technologies are explained in details of following chapters of Part A of the Manual as follows:
- Conventional Water Treatment processes: Chapter 8
- Disinfection Options: Chapter 9
- Specific Water Treatment Processes/Technologies: Chapter 10

iv. Additional units may be added (in case of high turbidity, colour etc.) suiting to the raw water quality as per the local conditions.

v. *Biological Activated Carbon (BAC) is Granular Activated Carbon (GAC) only. BAC is an operational mode. When Granular Activated Carbon is operated in Biological active mode, it is termed as BAC. In BAC mode, one does not back wash the filter with chlorinated water like normal filters. This is done to avoid loss of bacterial growth in the GAC bed. So in BAC mode the filter is backwashed with non-chlorinated water. In BAC mode, it is more effective in removing low BOD or Natural Organic Material (NOM)/ TOC through bacterial metabolism.

vi. Treatment options as suggested at S. Nos. 5 and 6 should only be considered if no alternate and economically sustainable water source is available.

8.1.4 Groundwater with High TDS

In many costal zones in India, the groundwater has TDS in the range of 1,000 to 5,000 mg/L (brackish to saline). In such cases, low-pressure RO membranes need to be included in the treatment process.

8.1.5 Conventional Water Treatment Options

The unit operations in water treatment include aeration, flocculation (rapid and slow mixing), clarification, filtration, disinfection, water conditioning, and many different combinations of these to suit these requirements. The treatment technologies for removal of emergent contaminants and DBPs have been included in Part A – Chapter 10 of this manual.

The choice of any particular sequence of treatment units will depend not only on the qualities of the raw water available and treated water desired but also on the comparative economics of alternative treatment steps applicable.

In the case of groundwater and surface water with storage that is well protected, where the water has turbidity below 10 NTU, and they are free from odour and colour, plain disinfection by chlorination is adopted before supply.

Where groundwater contains excessive iron, dissolved carbon dioxide, and odorous gases, aeration followed by flocculation (rapid and slow mixing), sedimentation, rapid gravity or pressure filtration, and disinfection may be necessary.

In case it contains only carbon dioxide or odorous gases, aeration followed by disinfection may be sufficient.

In surface waters with turbidity not exceeding 50 NTU and where a sufficient area is available, plain sedimentation followed by slow sand filtration and disinfection is practised mostly in rural areas.

Conventional treatments including pre-chlorination, aeration, flocculation (rapid and slow mixing) and sedimentation, rapid gravity filtration, and post-chlorination are adopted for polluted surface waters laden with algae or other microorganisms.

Sometimes, unconventional treatment may be adopted for waters of low turbidity (below 10 to 15 NTU) and containing a low concentration of suspended matter (less than 50 mg/L), Such raw waters are applied to the rapid sand filters with alum addition accompanied by slow mixing for a short period (10 min.).

However, due to lack of reliable classification of raw water sources (quality), full-fledged treatment is adopted at most of the places.

Water with excessive hardness needs softening or demineralisation by ion exchange.

8.1.6 Plant Capacity and Hydraulic Overloading

Hydraulic loading is defined as the volume of raw water or the water (design flow for next 30 years from the base year) applied to the unit operation per time period.

The treatment plant needs to be designed for 20% hydraulic overloading (not the process units). The interconnecting piping, channels and conduits are designed to carry 20% excess over the design flow. It also means establishing the hydraulic gradient of the treatment plant so that the desired freeboard is maintained in the open channels and process units. However, if the raw water source is subjected to high turbidity during flash floods, coagulant/coagulant aids dose may be increased proportionately for such flash period, limiting hydraulic overloading to 20% and diverting excess hydraulic overloading to maintain economy as the interconnections and channels are designed for 20% hydraulic overloading only.

Generally, the plant may be categorised based on the capacity as given in Table 8.3.

Table 8.3: Size of the Plant

S. No.	Size of WTP	Capacity
1.	Small	Less than 5 MLD
2.	Medium	5-25 MLD
3.	Large	More than 25 MLD

8.2 Pre-Sedimentation and Storage

The turbidity of raw water from rivers and streams may exhibit wide fluctuations, and values exceeding 500–1,000 NTU are not uncommon during high flow season (monsoon). The river's sediment load during floods chiefly derives from soil erosion and consists predominantly of coarse suspended solids. In North India, snowmelt in summer can cause high silt loads. Pre-sedimentation and storage can accomplish the removal of large-sized and rapidly settleable silt and other materials before the raw water reaches the treatment plant.

For pre-sedimentation, detention periods of two hrs. to four hrs. have been recommended. These plain sedimentation tanks can be constructed with conventional construction material or dug out of the earth with sloping sides. At least two tanks are provided if the settled solids removal is manual.

Unlike pre-settling basins, the storage basins or reservoirs are designed for very large detention periods ranging from about one week to a few months. The storage basins or reservoirs are proposed when the source is canal water (to take care of the canal closure period). They are also provided in the coastal region, where the tides affect the salinity of rivers. The storage basins also help reduce raw water's turbidity and suspended solids.

8.3 Aeration

In water treatment, aeration is practised for four purposes:

a) To enhance the aesthetic purpose or value of the water treatment complex as a whole for surface water as well groundwater. Aeration techniques are commonly used in pond, lake, and reservoir management to address low oxygen levels or algal blooms.
b) To add oxygen to water for imparting freshness, e.g., water from underground sources devoid of or deficient in oxygen so as to maintain DO level of 6-8.5 mg/L.
c) Expulsion of carbon dioxide, hydrogen sulphide, and other volatile substances, including volatile organic compounds (VOC), causing taste and odour.
d) Oxidation of iron, manganese, etc., from groundwater (water from deeper layers of rocks formation when contain iron, manganese, etc.). Iron and manganese removal improves with higher pH; therefore, aeration itself, which often raises pH due to stripping of CO_2, improves iron and manganese removal. Between iron and manganese, iron oxidises better at lower pH.

8.3.1 Types of Aerators

Following is the list of various types of aerators that design engineer should consider and apply an appropriate type of aerator based on the raw water quality and site-specific conditions while understanding the pros and cons of each of the types of the aerators.

8.3.1.1 Spray Aerators

Water is sprayed, through nozzles, upward into the atmosphere and broken up into either a mist or droplets. Water is either directed vertically or at a slight inclination to the vertical. The installation consists fixed nozzles on a pipe grid with necessary outlet arrangements.

Nozzles usually have diameters varying from 10 mm to 40 mm spaced in the pipe at intervals of 0.5 m to 1 m or more. Special (patented) types of corrosion-resistant nozzles and sometimes plain openings in pipes, serving as orifices, are used. The pressure required at the nozzle head is usually 7 m of water (2 to 9 m), and the discharge ratings per nozzle vary from 18 to 36 m^3/hr. Usually, aerator area of 0.03 to 0.09 m^2/m^3/hr. of design flow is provided.

The diameters of the pipe grid are very important and orifices should be so designed as to ensure a uniform discharge (with a maximum variation of 5%) through all the nozzles in the grid. The loss of head in the pipe is kept low compared to the loss of head in the nozzle. Theoretically, numerous small nozzles capable of producing atomised water could be used. Practically, however, extremely small nozzles are to be avoided because of clogging and consequent excessive maintenance needed. Common friction formulae are used in the estimation of loss of head, excepting that the pipe with nozzles has to be considered to be carrying uniformly decreasing flow.

8.3.1.2 Waterfall or Multiple Tray Aerators

Water is discharged through a riser pipe and distributed onto a series of trays or steps from which the water falls either through small openings to the bottom or over the edges of the trays. Water is caused to fall into a collection basin at the base. In most aerators, coarse media such as coke, stone, or ceramic balls, ranging from 50 to 150 mm in diameter, are placed in the trays to increase efficiency. For iron removal (see 9.4.3), this may be beneficial. The trays, about 4 to 9 in number (with a spacing of 300 mm to 750 mm), are arranged in a structure 1 m to 3 m high. With the media, good turbulence is created, and large water surface is exposed to the atmosphere. By the addition of more trays, the time of contact can be increased. The space requirements vary from 0.013 to 0.042 m^2 per m^3/hr. of flow. Natural ventilation or forced draft is provided. Removal efficiencies varying from 65% to 90% for CO_2 and 60% to 70% for H_2S have been reported.

8.3.1.3 Cascade Aerators

This is the most commonly used aerator for surface water treatment plants (Figure 8.1). In cascade aerators, water is allowed to flow downwards after spreading over an inclined surface in thin sheets, and the turbulence is secured by allowing the water to pass through a series of steps or baffles. The central shaft (inlet) of the aerator is circular in shape. It can be constructed in RCC or can be a pipe. The velocity in the shaft is limited to 0.60 m/sec to reduce the exit turbulence. The top opening of the shaft is provided with a metallic grille from a safety point of view. The number of steps is usually 4 to 6. Exposure time can be increased by increasing the number of steps, and the area-to-volume ratio can be improved by adding baffles to produce turbulence. Head requirements vary from 0.5 m to 3.0 m (optimum is 0.50 m to 1.20 m) and the space requirements vary from 0.015 to 0.045 m^2/m^3/hr. Generally, the 'rise' of the cascade is limited to 0.15 m to 0.25 m. The 'tread' to 'rise' ratio of cascades needs to be more than two to avoid the tendency of the water to 'jump over' instead of forming a thin film. In cold climates, these aerators must be housed with adequate provision for ventilation. Corrosion and slime problems may be encountered. The gas transfer efficiency is less compared to the spray type. Removal of gas varies from 20% to 45% for CO_2 and up to 35% for H_2S. Well-designed circular cascades aerators enhance the aesthetics of the treatment plant.

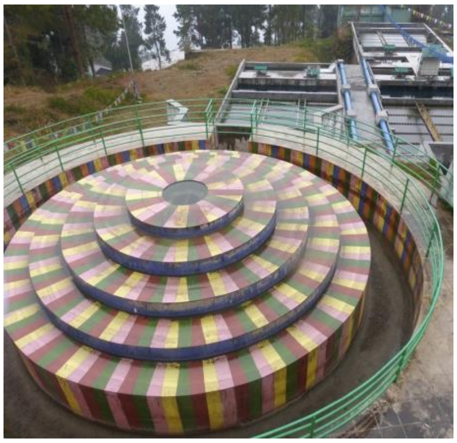


Figure 8.1: Cascade Aerators with Four to Six Steps

8.3.1.4 Diffused Aerators

This type of aerator consists of a basin in which perforated pipes, porous tubes, or plates are used to release fine bubbles of compressed air, which then rise through the water being aerated. As the rising bubbles of the air have a lower average velocity than the falling drops, a diffused air type provides a longer aeration time than the waterfall type for the same power consumed. These have higher initial costs and require greater recurring expenditure. Tanks are commonly 3 m to 4.5 m deep and 3 m to 9 m wide. Compressed air is injected through the system to produce fine bubbles, which on rising through the water, produce turbulence resulting in a continual change of exposed surface. Ratios of width to depth should not exceed 2:1 for effective mixing and the desired detention period varies from 10 min. to 30 min. The amount of air required ranges from 0.06 m^3 to 1 m^3 of air per m^3 of water treated. The air diffusers are located on one side of the tank. The power requirements of the blower vary from 3 to 13 w/m^3/hr.

The air should be filtered before passing through porous diffusers, and an oil trap is also provided before diffusers. Diffused aerators require less space than spray aerators but more than tray aerators, and cold weather operating problems are not encountered. The aerators can also be used for the mixing of coagulants. The designer is required to appropriately size the air compressor.

8.4 Measurement of Flow

The measurement of flow in open channel is very important in operating the various process in water treatment plant to monitor and control the process. The various methods used are explained in the following section.

8.4.1 Triangular Notches or V-Notch

There are generally three types of triangular notches used for flow measurement, i.e., 30°, 60° and 90°. 90° triangular notches are used for measuring small quantities of flows up to about 1.25 m^3/s.

i. **Installation Requirements**
   The approach channel should be reasonably smooth, free from disturbances and straight for a length equal to at least 10 times the width. The structures in which the notch is fixed shall be rigid and watertight and the upstream face vertical. The downstream level should be always at least 5 cm below the bottom-most portion of the notch (inverted apex) ensuring free flow.

ii. **Specification for Materials**

The plate should be smooth and made of rust-proof and corrosion-resistant material. The thickness should not exceed 2 mm, with the downstream edge chamfered at an angle of not less than 45° with the crest surface.

iii. Measurement of Head Causing the Water Flow

The head causing flow over the notch shall be measured by standard hook gauge upstream at a distance of three to four times the maximum depth of flow over the notch.

iv. Limitations

The triangular notches should be used only when the head is more than 60 mm.

v. Accuracy

The values obtained by the equation for triangular notches would vary from 97% to 103% of the true discharge for discharges from 0.008 m^3/s to 1.25 m^3/s.

8.4.2 Rectangular Notches

The installation requirements, specifications, head measurements, head limits and accuracy will be the same as for triangular notches. The width of notch should be at least 150 mm.

There are two types of rectangular notches, viz., (i) with end contractions and (ii) without end contractions.

i. With End Contractions

The contraction from either side of the channel to the side of the notch should be greater than 0.1 m.

The discharge (m^3/s) through a rectangular notch with end contractions is given by the equation:

$$Q = \frac{2}{3} E C_e \sqrt{2g}\, b_e\, H^{1.5}$$

Where:

b_e = effective width = actual width of the notch + k (value of k being 2.5 mm, 3 mm and 4 mm for b/B ranges of up to 0.4, 0.4 to 0.6 and 0.6 to 0.8 respectively);

b/B = ratio of the width of the notch to the width of the channel;

$H^{1.5}$ = effective head = actual head measured (h) + 1 mm;

g = acceleration due to gravity (9.806 m/s^2); and

C_e = varies from 0.58 to 0.70 for values of b/B from 0 to 0.8.

ii. Without End Contractions

The discharge (m^3/s) through a rectangular notch without end contractions is given by the following expression:

$$Q = \frac{2}{3} C_e \sqrt{2g} b H^{1.5b}$$

Where:

b = width of the notch (m)

H = effective head = actual /measured head (h) + 1.2 mm

C_e = 0.602 + 0.075 h/p

p = height of the bottom of the notch from the bed of the channel

8.4.3 Parshall Flume

Parshall flume is a type of standing wave flume widely used. However, for accuracy similar to other flumes types, its use requires application of different equations based on the throat size. The important thing to be observed is that the length of approach channel should be 10 times the 'throat width'. The length of downstream channel should be seven times the 'throat width' of the flume.

IS: 14371-1996 prescribes various methods to be adopted for measurement of flow of water in open channels through Parshall flume in water treatment plant.

Simplified formulae of measurement of flow in open channels

90° V-notch : $Q = 1.38\ H_w^{5/2}$

Rectangular Weir/notch : $Q = 1.84\ B\ H_w^{3/2}$

Parshall Flume : $Q = 2.27\ W\ H_a^{3/2}$

(Source: Shulz and Okun: Surface Water Treatment for communities in Developing Countries)

Where:

Q = Discharge (m^3/sec)
H_w = Head on weir (m)
H_a = Depth at entrance to the flume at specified measuring point (m)
B = Length of the weir (m)
W = width of throat (m)

The below listed IS codes of practice describes various methods of flow measurement which may be referred to,

a) IS 16698: 2019 Hydrometry — Selection, Establishment and Operation of a Gauging Station
b) IS 1192: 2013 (Reaffirmed Year: 2018) Hydrometry - Measurement of Liquid Flow in Open Channels Using Current-Meters or Floats.
c) IS 6330 (2012, Reaffirmed Year: 2022): Liquid Flow Measurement in Open Channels by Weirs and Flumes - End Depth Method for Estimation of Flow in Rectangular Channels with a Free Overfall (Approximate Method), which specifies a method for the estimation of sub-critical flow of clear water in smooth, straight, rectangular prismatic open channels with a vertical drop and discharging freely. Using the measured depth at the end, the flow in rectangular channels (horizontal or sloping) with confined nappe and unconfined nappe may be estimated.
d) IS 9108: 2020 Hydrometry – Open Channel Flow Measurement Using Thin-Plate Weirs (Second Revision)
e) IS 13083: 2017 (Reaffirmed Year: 2022) Hydrometric Determinations – Flow Measurement in Open Channels Using Structures – Flat -V Weirs (First Revision)
f) IS 14673: 2022 Hydrometry-Open Channel Flow Measurement Using Triangular Profile Weirs
g) IS 14869: 2016 (Reaffirmed Year: 2021) Flow Measurement Structures - Rectangular, Trapezoidal and U-Shaped Flumes (First Revision)
h) IS 14974: 2018 Hydrometry – Open Channel Flow Measurement Using Rectangular Broad-Crested Weirs (First Revision)
i) IS 14975 (2001, Reaffirmed Year: 2022): Measurement of Liquid flow in Open Channels – Streamlined Triangular Profile Weirs which specifies methods for the measurement of the flow of water in open channels under steady flow conditions using streamlined triangular profile weirs.
j) IS 15123 (2002, Reaffirmed Year: 2022): Hydrometric Determinations – Flow Measurement in Open Channels Using Structures – Trapezoidal Broad-Crested Weirs which specifies a

method of steady flow measurement in open channels using a trapezoidal broad-crested weir under modular and non-modular conditions.

k) IS 15353 (2003, Reaffirmed Year: 2018): Liquid Flow Measurement in Open Channels by Weirs and Flumes – V-Shaped Broad-Crested Weirs which specifies a method for the measurement of sub-critical flow in small rivers and artificial channels using V-shaped broad-crested weirs.

8.4.4 Instruments – Flow Indicators and Recorders

8.4.4.1 Simple Calibrated Scale

Simple calibrated scale is a device which indicates the depth of flow which can be used further to calculate the discharge with the help of some calibrated equations.

8.4.4.2 Float and Dial Type Indicator

In a water supply system, water tanks are used at different stages requiring level measurements in the tanks. Float and dial type level indicator is used for measuring the level in fluid tanks such as water tanks and sewage tanks. It consists of a float, tied to a SS wire rope other side of which is wound on a drum carrying constant torque spring to maintain the rope under continuous tension. Due to change in fluid level, the float rises or falls and rotate the drum. This motion is transmitted through gear mechanism to a pointer moving over a calibrated dial to display level in meters. Digital/analogue output can also be provided. Float type level indicators are simple construction and are easy to install. They are very sturdy and require very less maintenance. In water treatment plants, a small chamber is constructed on the upstream side of weir/notch or a flume to place the float in that. The upstream level is transferred to the float chamber by means of a small pipe at bottom.

8.4.4.3 Mechanical Integrator

The ball-and-disk integrator is a key component of many advanced mechanical integrators. A float is attached to the input carriage, so the bearing moves up and down with the level of the water. As the water level rises, the bearing is pushed farther from the centre of the input disk, increasing the output's rotation rate. By counting the total number of turns of the output shaft (for example, with an odometer-type device), and multiplying by the cross-sectional area, the total amount of water flowing past the metre can be determined. It is easy to maintain and operate.

8.4.4.4 Ultrasonic Flowmeter

The flow is measured based on the ultrasonic pulse which is emitted and received by transducers. The transit times of the pulses depends on the velocity of the fluid through which it passes. These transit times are measured, and their difference is proportional to the fluid flow rate. The disadvantage of this flowmeter is that the flow rate depends on a cross-sectional velocity profile.

8.4.4.5 Electromagnetic Probe Method

When an electromagnetic probe is immersed in flowing water, a voltage is created around the probe. This voltage, sensed by electrodes imbedded in the probe, is transmitted through the cable to the metre box. The voltage created by water flowing through the magnetic field is proportional to the velocity of flow of water. These small voltages are electronically processed and displayed on the panel metre.

Details of various metres can be referred from Chapter 13 in Part A of this manual.

8.5 Coagulation and Flocculation

'Coagulation' describes the effect produced by the addition of a coagulant and coagulant aids to raw water, resulting in particle destabilisation and charge neutralisation. This is achieved by rapid intense mixing in a unit called as a ‘Flash Mixer’ or ‘Rapid Mixer’ for obtaining uniform dispersion of the coagulant.

'Flocculation' is the second stage formation of settleable particles (or flocs) from destabilised colloidal sized particles occurs. This is achieved by gentle and prolonged mixing which ensures continuous multiple re-contacts of solids.

8.5.1 Rapid Mixing (Options for Coagulation)

The coagulant, coagulant aids and/or pH adjustment chemicals are normally introduced at some point of high turbulence in the water. The sources of power for rapid mixing to create the desired intense turbulence are gravitational, mechanical, static, and pneumatic mixers.

Where head loss through the plant is to be conserved as much as possible and where the flow exceeds 300 m^3/hr., mechanical mixing also known as flash mixing, is desirable. For larger plants, multiple units must be provided. Normally a detention time of 30 to 60 seconds is adopted in the flash mixer. Head loss of 0.2 m to 0.6 m of water, which is approximately equivalent to 1 watts/m^3 to 3 watts/m^3 of flow per hour, is usually required for efficient flash mixing. Gravitational or hydraulic devices are simple but not flexible, while mechanical mixers or pneumatic devices are flexible, but require external power and maintenance of rotary parts except for static mixers.

The IS 7090 (1985, Reaffirmed 2001): Guidelines for rapid mixing devices may be referred to for details for design considerations, guidelines for materials and methods of construction of the different types of rapid mixing devices:

a) Gravitational or Hydraulic:

- i) Hydraulic jump/Weir mixers,
- ii) Baffled channel, and
- iii) In-line mixers Gravitational or Hydraulic Devices

b) Mechanical:

- i) Impeller/Propeller type mixer, and
- ii) Pneumatic type mixer

a) Gravitational or Hydraulic:

In these devices, the required turbulence is obtained from the flow of water under gravity or pressure. Some of the more common devices are described below.

(i) Hydraulic Jump Mixing

This is achieved by a combination of a chute followed by a channel with or without a sill. The chute creates super critical flow (velocity 3 m/s to 4 m/s), the sill defining the location of the hydraulic jump and the gently sloping channel induces the jump. Standing wave flumes specially constructed for measurement of flow can also be used in which the hydraulic jump takes place at the throat of the flume.

(ii) Weir Mixer

Sudden drop in hydraulic level of water over a weir can cause turbulence and coagulants can be added at this ‘plunge’ point with the aid of diffusers (Figure 8.2 & 8.3). This is a rectangular weir or a notch. A chamber of 1 min. detention time is provided on the upstream site of the weir. A freefall of 0.5 m to 0.60 m on the downstream side creates sufficient turbulence (G value 800 to 1000 sec^{-1}) for instantaneous mixing of raw water with coagulants. A small chamber of detention time of 5 to 10 seconds is provided on the downstream side of the thorough mixing area.

Figure 8.2: Weir Mixer (Rapid Mix Unit)

Figure 8.3: Rapid Mix Weir showing Rectangular Weir, Mixing Basin, Coagulant Feeder Pipe

(iii) Baffled Channel Mixing

In this method, the channel section (neglecting the baffle) is normally designed for a velocity of 0.6 m/s. The angle subtended by the baffle in the channel is between 40° to 90° with the channel wall. This angle should ensure a minimum velocity of 1.5 m/s while negotiating the baffle.

(iv) In-Line Mixers

In pressure conduits, the coagulants can be added at the throat of a venturi or just upstream of orifice located within the pipe. In this system, no effective control is possible even though mixing takes place. Rapid mixing can also be obtained by injection of coagulants preferably, in the suction end or delivery end of low lift pumps where the turbulence is maximum.

Mechanical Devices

(i) Propeller Type Impellers/Turbines

These are commonly employed in flash mixers, with high revolving speeds ranging from 100 to 1400 rpm or more. The blades are mounted on vertical or inclined shaft and generate strong axial currents. Turbine types and paddle types are also used. In the design of a mechanical flash mixer unit, a detention time of 30 s to 60 s. is provided. The relatively high-powered mixing devices should be capable of creating velocity gradients of 300 s^{-1} (m/sec/m) or more. Power requirements are ordinarily 1 to 3 watts per m^3/hr. of flow.

(ii) Pneumatic Devices

When air is injected or diffused into water after suitable compression, it normally expands isothermally, and the resultant work done by the air can be used for necessary agitation. They are not common in water works practice. The typical range of velocity gradients and contact times are in the range of 3,000 s^{-1} to 5,000 s^{-1} and 0. 5 s to 0. 4 s. respectively.

8.5.1.1 Location of Coagulant Dosing Points

The primary coagulant is dosed just before the zone of maximum velocity gradient (turbulence) in the tank. For weir mixers, it is located over the 'nappe' of a free-falling weir. In the mechanical rapid agitator tank, it is near to the inlet of the tank. In the weir mixer, the coagulant aid is dosed about 8 to 10 seconds in the downstream mixing channel. In the mechanical agitator, the coagulant aid is dosed near the outlet of the tank (or below the agitator) in the diminishing zone of velocity gradient. Lime, if dosed to supplement the alkalinity deficiency in the raw water, is introduced prior or along with the primary coagulant. Typical Dosing point locations in the WTP with Flash Mixer are shown in Figure 8.4.

Figure 8.4: Dosing point locations in the WTP with Flash Mixer

8.5.1.2 Undesirable Dosing Practices

Dumping of alum blocks directly into the elevated channel or mixing channel must be totally avoided and is undesirable. The alum mixing and dosing becomes non-uniform resulting in a deteriorated treated water quality as shown in Figure 8.5

Figure 8.5: Direct dumping Alum block (Not Recommended)

8.5.2 Chemical Solution Feed

Preparation of the solution of the coagulant in the water of desired strength is the first step and is done in the solution tanks. This solution is fed to the raw water through controlled feeders which are of gravity or pressure type. This selection of the proper type of feeders and the point of application are important. Also, as different coagulants are to be fed at different points, the location at which the coagulants are fed is important to derive maximum mixing efficiency.

8.5.2.1 Solution Tanks

There should be at least two tanks for each coagulant feed. The capacity of each tank should generally be such as to hold 12 hours requirement at the maximum demand of coagulant at the design flow. If three tanks are provided, each should have a capacity of 8 hours requirement. A minimum freeboard of 0.3 m is necessary. Dissolving trays or boxes and also adequate facilities for draining the solution tanks should be provided.

The coagulant solution tanks should be located in or as near the coagulant storage area as possible to avoid unnecessary lifting and handling of coagulants. These tanks should preferably be located at a suitable elevation to facilitate gravity feed of the coagulant solution. However, pressure feed is recommended for proper dosing and mixing.

8.5.2.2 Preparation of Solutions

It is essential to ensure that all the solid coagulants are dissolved before the solution is put into raw water and the homogeneity of the prepared coagulant solution is maintained. This can be achieved by proper mixing by mechanical agitation. Manual mixing may be adopted for very small plants with capacities not exceeding 3.0 MLD. The drive mechanism shall be located on the top of the tank. The agitator shaft and paddles shall be of SS 304/316 material.

8.5.2.3 Solution Feed Devices

Solution feed devices are used to regulate the doses of coagulants fed into water. The rate of flow of the coagulant solution of known strength is prepared in the solution tank and is measured by means of either an orifice rota metre, positive displacement pump, or weirs. The solution feed equipment should be simple in operation and corrosion resistant.

8.5.2.4 Solution Feeders

There are several types of solution feeders, some of which are discussed below:

(a) Pot Type Coagulant Feeders

The pot type coagulant feeder is a simple type of equipment for feeding alum or alkali chemicals into water. The coagulant, in large crystal or lump form, is deposited into the feeding pot. A special orifice fitting, placed in the raw water line, contains an orifice plate which creates a pressure differential in pipes which connect the coagulant pot into the orifice fitting.

This pressure differential causes a small stream of water to flow from the high pressure side of the orifice plate through a pipe and a regulating valve, into the bottom of the coagulant feeding pot and this forms an equivalent stream of the coagulant solution, formed in the pot, to flow out of the top of the pot into the raw water line on the low-pressure side of the orifice plate.

(b) Pressure Solution Coagulant Feeders

Pressure solution coagulant feeders are much more accurate than the pot type coagulant feeders. In these, a coagulant solution of a definite strength is made by dissolving a weighed amount of coagulant in a specified volume of water in the coagulant solution tank. This batch of coagulant solution, when required, is charged into the displacement tank through the bottom. As the specific gravity of the coagulant solution is higher than that of water, the water in the displacement tank is displaced upwardly to waste through a valve.

A sight glass at the side of the feed tank has in it a glass float, which is so constructed that it floats in the heavy coagulant solution but sinks in water. This float indicates, at all times, the level of the coagulant solution thus notifying the operator when recharging is necessary.

(c) Electro-Coagulant Feeders

The water flows through an integrating raw water metre causing an electrical circuit to start the feed control unit through a time switch. The feed control unit is a mechanism designed to lower the swing draw off pipe at a rate which is proportional to the rate of flow of raw water. It consists of a motor, a speed reducing mechanism, two drums on which separate tapes are wound, a manual rewinding mechanism, a switch for operating an alarm for stopping the feed at low level in the solution tank and a dial for indicating directly the depth of solution removed from the tank.

(d) Gravity Orifice Coagulant Feeders

The gravity orifice coagulant feeder is limited in application to those cases where the flow rate of the water being treated is constant. The solution from the coagulant solution tank flows by gravity, through a strainer and through a float valve, into the orifice box. Gravity Orifice & Taper valve dosing Box and Gravity feed dosing box with V notch are shown in Figure 8.6 and 8.7 respectively.

(e) Reciprocating/Positive Displacement Pump Coagulant Feeders

This method of feeding coagulant employs a motor-driven reciprocating coagulant pump. The pump withdraws a coagulant solution, or suspension of suitable strength, from a tank and discharges the solution or suspension to the point of application under any desired pressure. The feeding pump may be designed to treat either a variable or a constant flow of water. This is the most desired way of feeding coagulants, coagulant aids, and/or pH adjustment chemicals.

Figure 8.6: Gravity Orifice & Taper valve Dosing Box

Figure 8.7: Gravity Feed Dosing Box with V-Notch

(f) Variable Rate Proportional Feeders

If the rate of flow of water being treated varies, proportional feeding of coagulants is necessary and is a preferred method. This is carried out by accurately measuring the amount of coagulant fed by the pump. This pump is a proportioning and metering device which delivers a definite volume of coagulant with each stroke. A water metre with an electrical contractor is placed in the raw water line. The contactor closes a circuit every time a given volume of water flows through the metre. The closing of the circuit energises the motor of the reciprocating pump, which then operates to deliver a given volume of coagulant until an electric time switch breaks the circuit, thereby stopping the pump. The cycle repeats itself approximately every 30 seconds, at maximum flow, with the pump operating for approximately 20 seconds after each contact. The amount of coagulant fed is thus accurately proportioned to the flow of water regardless of variations in the rate of flow, because both the volume of water treated between metre contacts and the volume of coagulant added to treat the water are accurately measured. However, this suffers from the disadvantage that, particularly when used with alum solutions, the water is subject to an overdose and no-dose sequence. It is better to have the coagulant pump run continuously and to modulate the stroke of the pump with a mechanical device. Reciprocating / Positive Displacement Pump Chemical Feeders are shown in Figure 8.8.

Figure 8.8: Reciprocating / Positive Displacement Pump Chemical Feeders

8.5.2.5 Dry Feed

Dry coagulant feeders incorporate a feed hopper mounted above the feeding device. This device may consist of a rotating table and scraper, a vibrating trough, or an oscillating displacer, or some equivalent method of moving the coagulant from the point where it leaves the feed hopper to the point of discharge. The rate of movement of the coagulant determines the quantity to be discharged on a volumetric basis. Gravimetric feeders are also available in which the quantity discharged in a unit of time is continuously weighed and the speed of operation automatically controlled to maintain a constant weight. The feeder may be designed for constant rate operation or for feeding coagulants in proportion to the rate of flow of water.

8.5.2.6 Coagulants

(i) Coagulants/chemicals used and their properties

Annexure 8.4 gives the list of coagulants and/or chemicals commonly used in water treatment and their properties. The engineer must carefully consider which is the most effective coagulant and/or chemical that should be used for the treating water based on site-specific conditions.

(ii) Coagulant Storage

The coagulant store should be of damp proof construction, properly drained. Special precautions against flooding should also be taken.

For coagulants purchased in bags, storage by piling at least 15 centimetres above the floor of the storeroom may be arranged. A height of stack not exceeding 2 m is recommended. Hygroscopic coagulants should be obtained in moisture-proof bags and stored in airtight containers.

All plants, particularly small ones (less than 5 MLD), should keep on hand at all times, a supply of coagulants sufficient to provide a safety factor. A storage of three months is advisable. Minimum storage of one month of monsoon requirement should be provided for larger plants.

Coagulants such as powdered activated carbon which are likely to cause dust problems should be stored in separate rooms.

Storage of acid materials near alkalis is undesirable as their contact generates considerable heat resulting in combustion. This is also true of oxidising coagulants such as chloride of lime mixed with activated carbon. Hence, they should be isolated. It is advisable to store chlorine cylinders separately as gaseous chlorine in contact with activated carbon leads to severe fire hazards.

8.5.3 Slow Mixing or Flocculation

Slow mixing is the hydrodynamic process which results in the formation of large and readily settleable flocs (orthokinetic flocculation) by bringing the finely divided matter into contact with the microflocs formed during rapid mixing. Alum is a trivalent coagulant (carrying +++ charges on its surface) and it nullifies three colloidal particles (each one carrying –ve charge) and therefore, Van der Waal forces of attraction increases, which forms the microflocs and later on, these microflocs again collide together to form a settleable floc. These flocs can be subsequently removed in settling tanks and filters.

8.5.3.1 Design Parameters

The desirable values of G in a flocculator varies from 20 s^{-1} to 75 s^{-1} and G.t from 2 to 6×10^4 for aluminium coagulants and 1 to 1.5×10^5 for ferric coagulants. The usual detention time provided, varies from 10 min. to 30 min. Very high G values tend to shear flocs and prevent them from building to size that will settle rapidly. Too low G values may not be able to provide sufficient agitation to ensure complete flocculation.

Another useful parameter is the product of G.t and the floc volume concentration 'C' (Volume of floc per unit volume of water). This parameter G.C.t reflects to a certain extent the contact opportunity of the particles, but the usefulness of this parameter is not yet fully established. The values are of the order of 100.

To ensure maximum efficiency in power input and to reduce possible shearing of particles during floc formation, tapered flocculation is sometimes practised. The value of G in a tank is made to vary from 100 in the first stage, to 50 or 60 in the second stage, and then brought down to 20 s^{-1} in the third stage in the direction of flow. Correction factors for Detention Time with respect to water temperature are given in Table 8.4.

Table 8.4: Correction Factors for Detention Time with respect to Water Temperature

Temp (° C)	Dynamic Viscosity (k) Ns/m^2	Detention Time factor
0	0.001792	1.35
5	0.00152	1.25
10	0.00131	1.15
20	0.001009	1.07
25	0.000895	0.95
30	0.000800	0.90
40	0.0006531	
50	0.0005471	

8.5.3.2 Types of Slow Mixers

Similar to rapid mixing units, these are categorised under gravitational or hydraulic, mechanical and pneumatic. The hydraulic type uses the kinetic energy of water flowing through the plant created usually by means of baffles, while mechanical type uses the external energy which produces agitation of water. Technologies that include a combination of rapid mix, flocculation, and clarification are very effective treatment process for raw water turbidity of 200 NTU or less and available land is limited. These technologies are known as upflow clarifiers, ContaClarifier, etc.

(i) Gravitational or Hydraulic Type Flocculators

Several types of gravitational or hydraulic flocculators are used in practice which are discussed as follows. All hydraulic flocculators require relatively less maintenance than mechanical flocculators.

(a) **Horizontal Flow Baffled Flocculator**

These flocculators consist of several around-the-end baffles with in-between spacing of not less than 0.45 m to permit cleaning (Figure 8.9 and 8.10). Clear distance between the end of each baffle and the wall is about 1.5 times the distance between the baffles, but never less than 0.6 m. Water depth is not less than 1.0 m, and the water velocity is in the range of 0.10 m/s to 0.30 m/s. The detention time is between 15 min. and 20 min. The flocculator is well suited for very small treatment plants. It is easier to drain and clean. The head loss can be changed as per requirement by altering the number of baffles. The velocity gradient can be achieved in the range 10-100 s^{-1}.

Figure 8.9: Horizontal Flow Baffled Flocculator

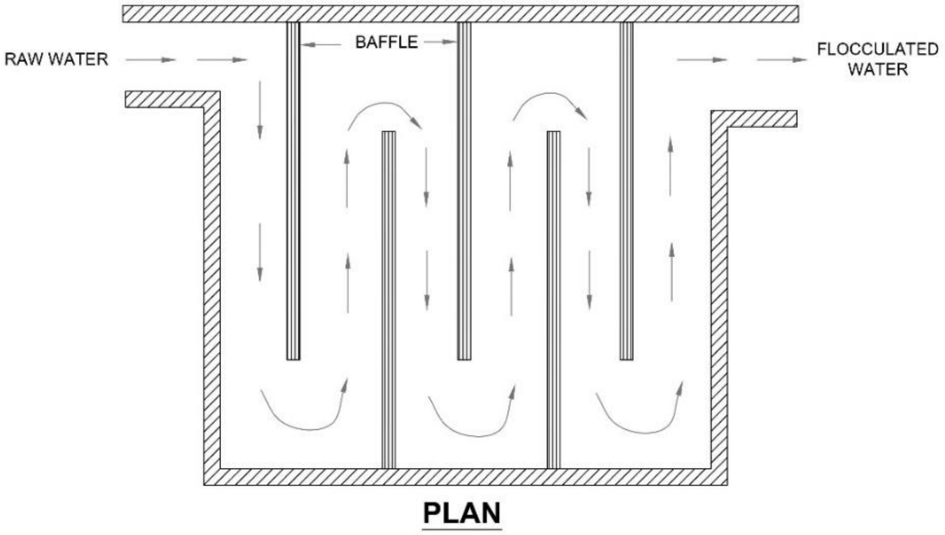


Figure 8.10: Plan of a Typical Horizontal Flow Baffled Flocculator

(b) **Vertical Flow Baffled Flocculator**

In these flocculators, the distance between the baffles is not less than 0.45 m (Figure 8.11). Clear space between the upper edge of the baffles and the water surface or the lower edge of the baffles and the basin bottom is about 1.5 times the distance between the baffles. Water depth varies between 1.5 to 3 times the distance between the baffles and the water velocity is in the range 0.1–0.2 m/s. The detention time is between 10-20 min. This flocculator is mostly used for medium- and large-sized treatment plants.

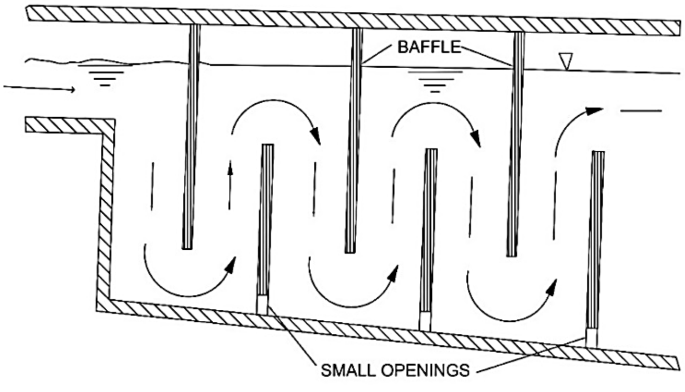


Figure 8.11: Vertical Flow Baffled Flocculator

(ii) Mechanical Paddle Type flocculator

Paddle flocculators are widely used in practice (Figure 8.12). The design criteria are: depth of tank is = 3 to 4.5 m; detention time is t = 10 to 40 min., normally 30 min; velocity of flow = 0.2–0.8 m/s, normally 0.4 m/s; total area of paddles = 10% to 25% of the cross-sectional area (length or width × depth) of the tank; range of peripheral velocity of blades = 0.2–0.7 m/s; (0.3–0.5 m/s is recommended); range of velocity gradient, G = 10 to 75 s^{-1} range of dimensionless factor Gt = 10^4 - 10^5 and power consumption; 10.0 to 36.0 kW/MLD, outlet velocity to settling tank where water has to flow through pipe or channel = 0.15 to 0.25 m/s to prevent settling or breaking of flocs.

It is desirable to provide more than one compartment in series to lessen the effect of short circuiting. It is desirable to provide a variable frequency drive (VFD) for paddle agitator to have flexible/adjustable G value as per the jar test.

Figure 8.12: Mechanical Type Vertical Flow Flocculator with Paddles

(iii) Pulsating Sludge Blanket Clarifier

In this type of clarifier, flocculation and clarification zones are superimposed. In flat bottom tanks, the flocculation takes place in the bottom part by using static mixers (Figure 8.13 and 8.14). In an already formed sludge blanket, coagulated water is injected in high flow rate for a very short period. As the 'pulsed' water passes uniformly through the sludge blanket, the blanket goes through the alternative cycles of expansion and contraction, thus dense floc is formed which has a high settling velocity. The clarified water is collected at the top. The excessive sludge generated is drained off periodically.

The advantages are that it is simple in construction and its adaptability to all shapes of tanks. As flocculation and clarification zones are superimposed, it produces far more compact plants. It has operational flexibility and placing of plates in the clarification zone further increases its efficiency. These types of clarifiers work well when raw water turbidity is low or moderate.

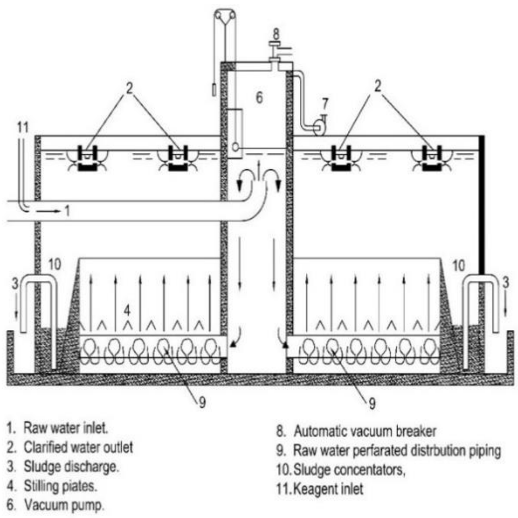


Figure 8.13: Pulsator Clarifier

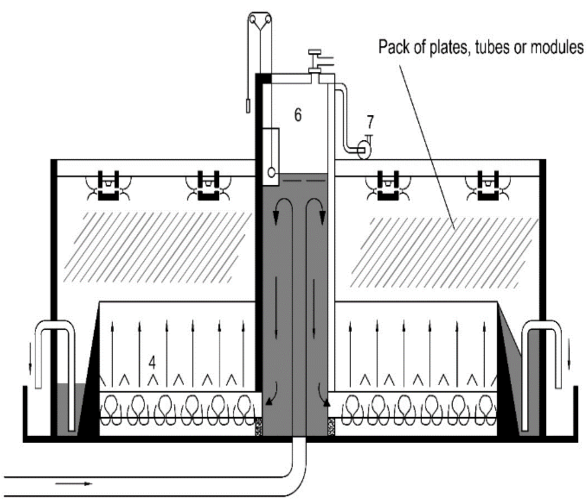


Figure 8.14: Superpulsator Clarifier

8.6 Sedimentation (Clarification)

Sedimentation is the separation of suspended particles from water by gravitational settling. It is one of the most commonly used unit operation of conventional water treatment. Sedimentation (settling or clarification) is used to remove coagulated impurities.

The factors that influence sedimentation are:

(i) size, shape, density, and nature (discrete or flocculent) of the particles;
(ii) viscosity, density, and temperature of water;
(iii) surface overflow rate;
(iv) velocity of flow;
(v) inlet and outlet arrangements;
(vi) detention period; and
(vii) effective depth of settling zone.

8.6.1 Types of Tanks

The tanks may be categorised into horizontal flow tanks or vertical flow tanks on the basis of direction of flow of water in the tank. The tanks may be rectangular, square, or circular in plan.

8.6.1.1 Horizontal Flow Tanks

In the design of a horizontal flow tank, the aim is to achieve as nearly as possible the ideal conditions of equal velocity at all points lying on each vertical line in the settling zone. The direction of flow in the tanks is substantially horizontal. Among the representative designs of the horizontal flow settling tanks, the following may he mentioned:

8.6.1.2 Radial Flow Circular Tank with Central Feed

The water enters at the centre of the tank and emanates from multiple ports of circular well in the centre of tank to flow radially outwards in all directions equally. The aim is to achieve uniform radial flow with decreasing horizontal velocity as the water flows towards the periphery and is withdrawn from the tank through effluent structure. The sludge is ploughed to central sump mechanically and continuously and is withdrawn during operation. The sludge removal mechanism consists of scraper blades mounted on two or four arms revolving slowly.

a. **Radial Flow Circular Tanks with Peripheral Feed:** These tanks differ from the central feed circular tanks in that the water enters the tank from the periphery or the rim. It has been demonstrated that the avenge detention time is greater in peripheral feed basins leading to better performance.

b. **Rectangular tanks** with longitudinal flow where the tanks are cut out of operation for cleaning. The solids are flushed to sump for removal from the dewatered tank.
c. **Rectangular tanks** with longitudinal flow where sludge is mechanically scraped to the sludge pit located usually towards the influent end and removed continuously or periodically without disrupting the operation of the tanks.

The BIS code 'IS 10313 (1982, Reaffirmed Year: 2021): Requirements for Settling Tank (Clarifier Equipment) for Water Treatment Plant', may be also referred to.

8.6.1.3 Vertical Flow Tanks

Vertical flow tanks normally combine sedimentation with flocculation. These tanks are square or circular in plan and may have hopper bottoms. The influent enters at the bottom of the unit where flocculation takes place as particles co-join into aggregates. The upflow velocity decreases with increased cross-sectional area of the tank. There is a formation of blanket of floc through which the rising floc must pass. Because of this phenomenon, these tanks are also called as upflow sludge blanket clarifier (Figure 8.15). The clarified water is withdrawn through circumferential or central weir.

These tanks have no moving parts and, except for a few valves, require no mechanical equipment. They are compact units requiring less land area.

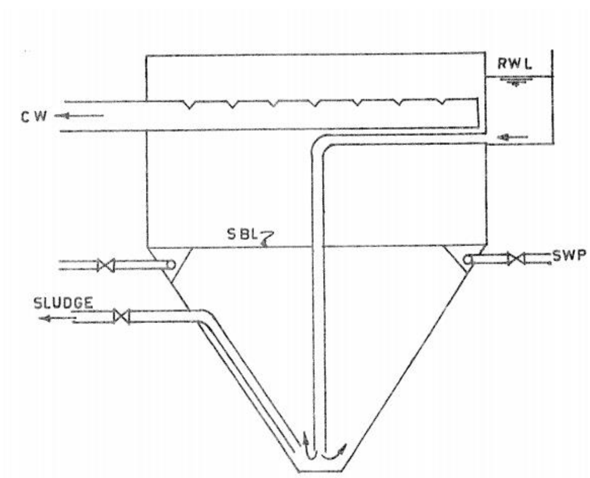


Figure 8.15: Sludge Blanket Clarifier

8.6.2 Clariflocculators and ContaClarifiers

Clariflocculators are widely used in India in water and wastewater treatment. The flocculation and sedimentation processes are effectively incorporated in a single unit in the clariflocculator.

The coagulated water enters into flocculation zone through the central shaft at the top. The velocity in the central shaft and that through outlet ports is restricted to 0.60 m/s.

8.6.2.1 ContaClarifiers or Upflow Contact Clarifiers

Upflow Contact Clarifiers can serve as a replacement for traditional coagulation, flocculation, and sedimentation in water treatment plants, saving on space and reducing maintenance on moving parts. The Upflow Contact Clarifier consists of an application specially sized and designed granular, non-buoyant media bed in three distinct sized zones, which is stable under all service flow conditions, an inlet distributor, air scour system, media support deck and screen, upper screen system, and effluent launder. Chemically dosed water is introduced to the bottom of the Upflow Contact Clarifier, as it flows through the multiple sections of media, interparticle collisions result in forced flocculation and floc formation. Once floc is formed, it then flows into the removal zone and turbidity and solids are greatly reduced through contact with the non-buoyant media and previously retained solids. Once terminal head loss is reached, a rinse cycle takes place, utilising raw water pushed at a higher velocity along with an air scour, excess retained particles are cleaned and removed from the Upflow Contact Clarifier. Typical loading rates for Upflow Contact Clarifiers range from 320–400 LPM/m^2, greatly increasing space efficiency over traditional sedimentation area.

8.6.2.2 Clariflocculators

Clariflocculators are widely used in India in water treatment (Figure 8.16, 8.17 and 8.18). The flocculation and sedimentation processes are effectively incorporated in a single unit in the clariflocculator. The unit has concentric circular flocculation zone at the centre and annular or peripheral clarification zone.

All these units consist of 2 or 4 flocculating paddles (slow agitators) placed equidistantly. These paddles rotate on their vertical axis. Their drive mechanisms are located on the walkway platform of rotating bridge. For plants less than 15 MLD capacity two agitators are sufficient, for higher capacities four are provided. The flocculating paddles may be of rotor-stator type rotating in opposite direction around this vertical axis. The clarification unit outside the flocculation compartment is served by inwardly raking rotating blades. The water mixed with chemicals is fed in the flocculation compartment fitted with paddles rotating at slow speeds.

The coagulated water enters into flocculation zone through the central shaft at the top. The velocity in the central shaft and that through outlet ports is restricted to 0.60 m/sec. The flocculator wall is supported on the equidistance columns. The flocculated water passes out from the bottom of the flocculation tank to the clarifying zone through the wide openings in between the supporting columns. The area of the opening being large enough to maintain a very low velocity (not more than 0.3m per minute). Under quiescent conditions in the annular settling zone the floc embedding the suspended particles settle to the bottom and the clear effluent overflows into the peripheral launder. The sludge which settles down to the bottom is continuously swept towards the central sludge pocket by the scrapper arms of rotating bridge. The slope of the tank bottom is in the range of 1:12 to 1:10 towards the center. It is advisable to locate the collection launder on the periphery outside the tank. If located inside, the projected area below the launder is not taken in to capacity calculations. The collection weir of the launder is provided in concrete (Ogee) with a sharp collection edge (like rectangular weir). Sometimes V notch weir plate is also provided as a collection weir. Generally, the maximum overall tank diameter is limited to 40 – 50m. The rotating bridge rests at one end on the bearings provided over the central shaft. At the other end the end carriage drive moves on the outer wall. Normally the wheel moves on the rail fixed on the wall. Sometimes rubber tyres are provided for the wheel. The tangential speed of scrapper rake tip is 2.5 to 3 m per minute. The side water depth of the clariflocculator is in the range of 3.5m to 4.0m. As was the practice of the past, it is not necessary to make provision for extra depth for storage of sludge in the clarification zone. For capacity calculations the volume of the conical bottom need not be taken in to consideration. For tank diameter up to 20m, sludge drain pipe diameter needs to be minimum 200mm, for larger tanks it needs to be minimum 300mm.

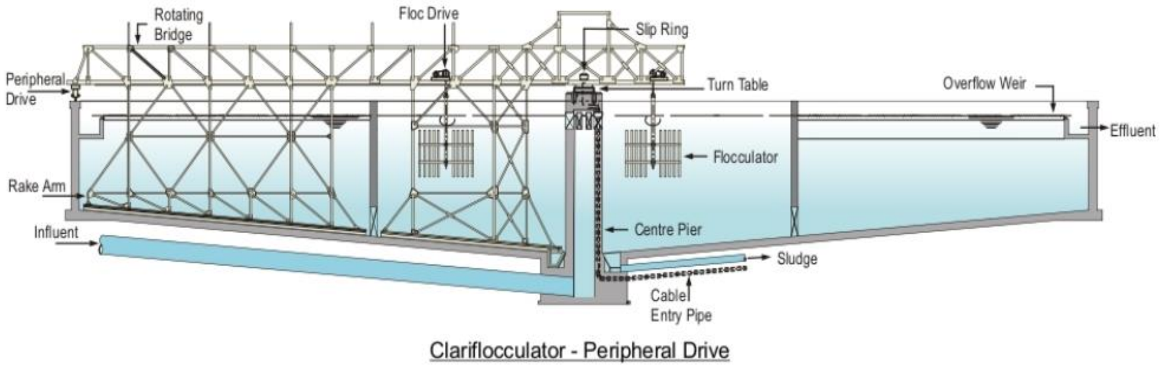


Figure 8.16: Cross section of Clariflocculator

Figure 8.17: Clarification zone with scrapper rotating bridge

Figure 8.18: Flocculation zone with slow agitators

8.6.3 Sedimentation Tank Dimensions

The length to width ratio of rectangular tanks should preferably be from about 3:1 to 5:1. The narrower the tank, the less chance there is for setting up of cross currents and eddies due to wind action, temperature changes and other factors involved. In very large-sized tanks where the depth is necessarily great, it may be advisable to provide longitudinal baffles to confine the flow to definite straight channels. These walls could be of thin sections since the pressure on both sides will be the same.

The diameter of the circular tank is governed by the structural requirement of the trusses that carry the scraping mechanism. Circular tanks up to 60 m in diameter are in use but are generally up to 30 m to reduce wind effects. Square tanks are generally smaller, usually with sides up to 20 m square tanks with hopper bottoms having vertical flow have sides generally less than 10 m to avoid large depths.

The depth of the settling basin depends on the character of sludge handled, storage capacity required and cost. In warm climates and where the sludge is likely to contain considerable organic matter, it is not advisable to store sludge for long periods; otherwise, the decomposition of the sludge adversely affects the settling process. Depths commonly used in practice very from 2.5 m to 5 m with 3.0 m being a preferred value. Bottom slopes are kept 1% or less in rectangular tanks and 1 in 12 or steeper for circular tanks. The slopes of sludge hoppers range from 1.2:1 to 2:1 (vertical: horizontal).

8.6.4 Common Surface Loadings and Detention Periods

The removal of particles of varying hydraulic subsidence values is solely a function of surface overflow rate also called ‘surface loading’ and is independent of the depth of the basin for discrete particles and unhindered settling. However, contact opportunities among particles leading to aggregation increase with increasing depths for flocculent particles having tendency to agglomerate while settling, such as alum and iron flocs. The range of surface loadings and detention periods for average design flow for different types of sedimentation tanks are given in Table 8.5:

Table 8.5: The range of Surface Loadings and Detention Periods for Average Design Flow for Different Types of Sedimentation Tanks

Tank type	Surface loading $m^3/m^2/d$*		Detention period, hr.*		Particles normally removed
	Range	Typical value for design	Range	Typical value for design	
Plain Sedimentation	Up to 60	15-30	1-15	3-4	Sand, silt, and clay
Horizontal flow, Circular/Clariflocculators	25-75	30-40	2-8	2-2.5	Alum and iron floc
Vertical flow (Upflow) Clarifiers	-	40-50	-	1-1.5	Flocculent

*At average design flow

8.6.5 Inlets and Outlets

Inlet structures must:

(i) uniformly distribute flow and suspended particles over the cross section at right angle to flow within individual tanks and into various tanks in parallel;
(ii) minimise large-scale turbulence; and
(iii) Initiate longitudinal or radial flow if high removal efficiency is to be achieved.

Water leaving the flocculator units should flow into the sedimentation basin through slots or effluent ports.

For uniform distribution of flow, the flow being divided must encounter equal head loss or the head loss between inlets on inlet openings must be small in comparison to the head available at the inlets.

Freely discharging weirs, anywhere between flocculator and settling tank, have a tendency to break fragile floc, hence, freefall is not recommended. Where water carrying floc has to pass through channels or pipes before reaching sedimentation basins, the velocity in such channels or pipes should be held between 0.15 to 0.25 m/s to prevent settling or breaking up of the floc.

The velocity of flow through such slots should be about 0.2 to 0.3 m/s, and head loss is estimated as 1.7 times the velocity head. The diameter of the hole should not be larger than the thickness of the diffuser wall.

To ensure uniform collection all along the periphery mild steel weir plate with 90º V-notches along with suitable weir clamps shall be provided. The V-notches are generally placed 150-300 mm centre to centre. A baffle is provided in front of the weir to stop the floating matter from escaping into effluent.

There is a growing trend towards the use of effluent launders or troughs covering a good part of the surface of the settling basins. These are spaced at a distance of one tank depth between the troughs. The use of maximum feasible weir length in the tank from the outlet towards the inlets assists greatly in controlling density currents. Weirs, however, suffer from the difficulty in levelling which is not faced with perforated pipe launder. Perforated launders, with ports commonly submerged 30 to 600 mm below the surface are useful in varying the water level in the basin during operation and prevent floating matter passing to the filters.

8.6.6 Weir Loading

Weir length relative to surface area determines the strength of the outlet current. Normal weir loadings are up to 300 m^3/d/m. But when settling tanks are properly designed, well clarified waters can be obtained at weir loadings of even up to 1,500 m^3/d/m. Finger weirs are also employed for Large Diameter Clarifiers/ Clariflocculators in case the weir loading rate exceeds the above range (Figure 8.19).

Figure 8.19: Finger weirs for Large Diameter Clarifiers/ Clariflocculators in case they exceed weir loading

8.6.7 Sludge Removal

Sludge is normally removed under hydrostatic pressure through pipes. The size of the pipe will depend upon the flow and the quantity of suspended matter. It is advisable to provide telescopic sludge discharge arrangement for easy operation and for minimising the wastage of water. For non-mechanised units, pipe diameters of 200 mm or more are recommended. Pipe diameters of 100 to 200 mm are preferred for mechanised units with continuous removal of sludge with hydrostatic head. In circular tanks, where mechanical scrapers are provided, the floor slopes should not be flatter than 1 in 12, to ensure continuous and proper collection of sludge. For manual cleaning, the slope should be about 1 in 10.

The power required for driving the scraping mechanism in a circular tank depends upon the area to be scraped and the design of the scraper. The scraping mechanism is rotated slowly to complete one revolution in about 30 min. to 40 min. or preferably the tip velocity of the scraper should be around 0.3 m/min. or below. Power requirements are about 0.75 W/m^2 of tank area.

Sludge and wash water should be properly disposed of without causing any problems of pollution if discharged into water courses.

For sludge blanket type vertical flow settling tanks, the slope of the hoppers should not be less than 55° to horizontal to ensure smooth sliding and removal of sludge. In such tanks special slurry weirs are provided with their crests in level with the top of sludge blanket for continuous bleeding of the excess sludge.

Special types of consolidation tanks with a capacity of 30 min. are sometimes provided to consolidate the sludge and recover water from it.

In non-mechanised horizontal flow rectangular settling tanks, the basin floors should slope about 10% from the sides towards the longitudinal central line adopting a longitudinal slope of at least 5% from the shallow outlet end towards the deeper inlet area where the drain is normally located. Manual cleaning of basins is normally done hydraulically, using high pressure hoses. Admitting settled water through the basin outlet helps this function. If sludge is to be withdrawn continuously or nearly continuously from the bottom of the basin by gravity without mechanical equipment, hopper bottoms have to be used with slope of not less than 55° to the horizontal.

Reclamation of water from the sludge removed from the settling basin should be encouraged. The various methods include disposal of sludge on land or on sludge drying beds.

8.6.8 Tube Settlers and Plate Settlers

Tube and plate settling devices provide excellent clarification with detention times of equal to or less than 10 min (Figure 8.20, 8.21, 8.22 & 8.23). Tube configurations are steeply inclined. In inclined tubes (about 60°) based on the ‘counter current’ flow principal, continuous gravity drainage of the settled sludge can be achieved.

While tube settlers have been used for improving the performance of existing basins, they have also been successfully used in a number of installations as a sole or independent settling unit. It has been found that if one-fifth of the outlet end of a basin is covered with tube or plate settlers, the effective surface loading on the tank is nearly halved or the flow through the basin can be nearly doubled without impairment of effluent quality.

The tubes may be square, hexagonal, diamond shaped, triangular, and rectangular or chevron shaped. The material of tubes is normally rigid PVC (other forms of plastics have also been used). Tubes can be extruded over the machine or alternatively fabricated out of thin plastic corrugated sheets (1.0 mm). Normally, black colour is preferred to prevent the growth of algae. In India, commonly, tubes with 50 mm × 50 mm (ID) section are practised. The raw material of tubes is virgin rigid PVC granules having specific gravity more than that if water.

Following Table 8.6 shows the surface loading rates to be adopted for different length of tubes (ready reckoner), assuming settling velocity of settling solids (floc) is 35 m/day (1.25 m/hr.)

Table 8.6: The Surface Loading Rates to be adopted for Different Length of Tubes

Tube Size (mm × mm)	Slant Length (mm)	Surface loading rate (on plan area)	
		$m^3/m^2/d$	$l/m^2/hr.$
50 × 50	600	123	5125
50 × 50	750	150	6250
50 × 50	1000	195	8125
Above 1,000 mm lengths are not recommended			

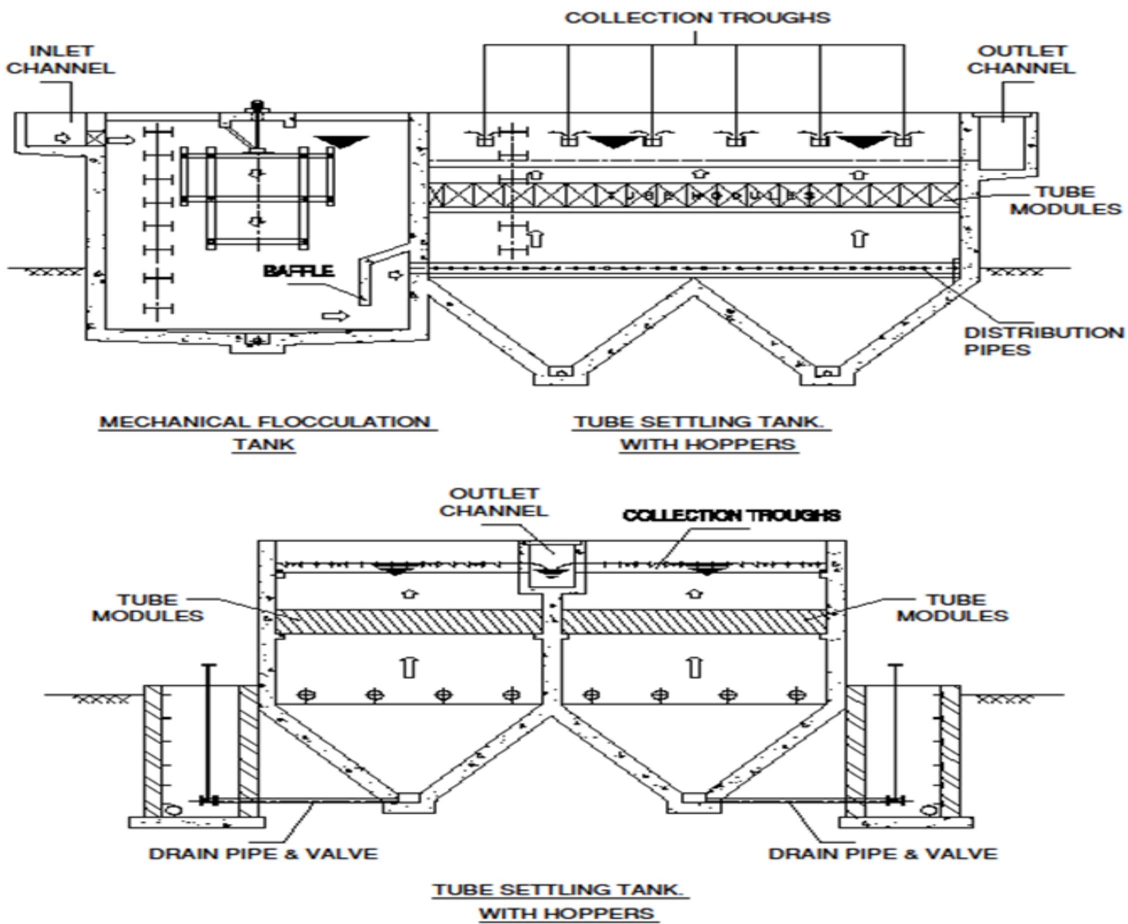


Figure 8.20: Typical Details of External Flocculation Tanks and Tube Settling Tanks with hopper bottom

Figure 8.21: Tube Settling and Flocculation under operation Modules & Troughs

8.6.8.1 Inlet and Outlet Considerations

Since the detention time affects the efficiency of tube settling tanks, inlet and outlet needs to be designed very carefully. The inlet velocity at the entry of the flocculated water needs to be less than the floc break velocity (0.1 m/s to 0.3 m/s). The entry ports need to be at least 1.0 m to 1.50 m below the bottom of the tube modules. The flocculated water is distributed uniformly with entry from one side. The outlet is in the form of uniformly spaced collection troughs with spacing varying from 1.0

m to 2.50 m. The troughs are located 0.60 m to 0.90 m above the top of the modules. The troughs ensure that uniform surface loading is imparted on the tubes.

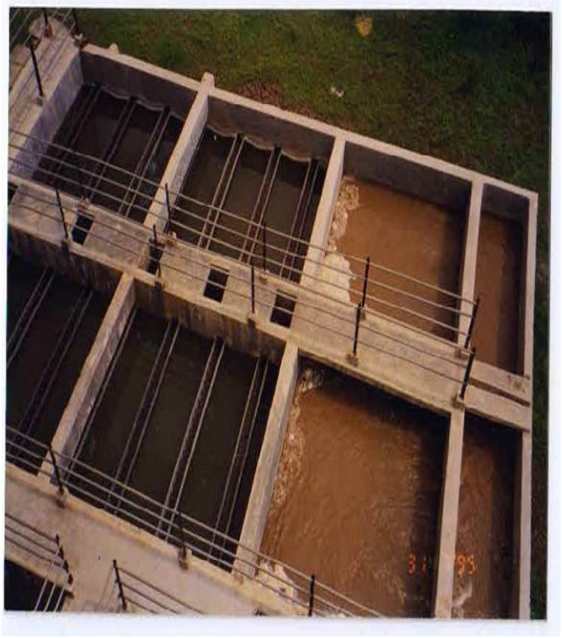


Figure 8.22: Performance photo at High Turbidity

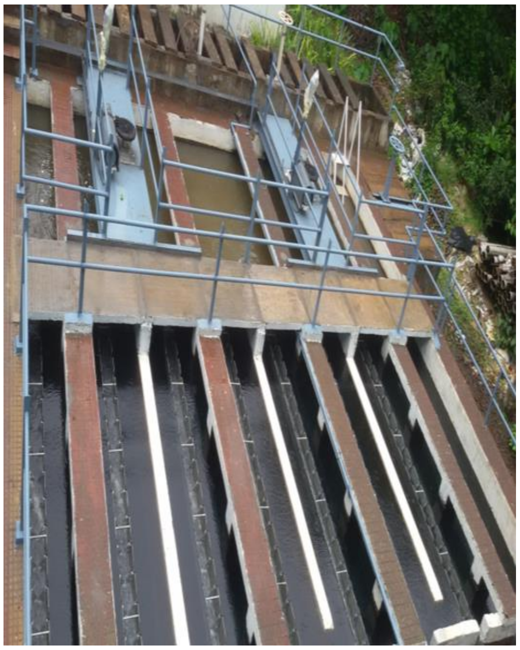


Figure 8.23: Performance photo at Low Turbidity

8.6.8.2 Design Parameters

For the above-described size and configuration of tubes, for steeply inclined tube settlers (60°) the surface loading rates (overflow rates) based on the plan area of the basin are recommended to be 5,000 l/h/m^2 to 6,500 l/h/m^2. These are three to four times higher than the conventional basins. The overall detention time of the tank should be 30 min. to 45 min. which is 25% to 40% of the conventional. The weir loading should be less than 300 m^3/m/d. These loading rates are applicable for average water temperatures of 15 °C to 30 °C. It is possible to achieve settled or clarified water turbidity of 5–10 NTU or even less for raw water turbidity range of 100–1000 NTU. The tube settling plants can be designed in square, rectangular, or circular configurations.

8.6.8.3 Sludge Removal

Tube settlers with multiple hopper bottom are recommended to be adopted for small and medium capacity plants up to 25–30 MLD for effective sludge removal. Hopper plan area is restricted to 4.5 m × 4.5 m at the top. The hopper walls are inclined at 45°. The drainpipe from each hopper (minimum 100 mm diameter) is laid horizontally from the hopper pit. The drain valve provided on pipe ensures periodical removal of sludge. For large plants, centrally driven scraper rake or rotating bridge scrapers are employed. The Spyder type sludge removal systems are very effective. The tube settling systems are also compatible with chain and flights scrapper mechanism.

8.6.9 Combination of Technologies

(a) **Tube – Clariflocculator (Clarisettler)**

Recent advances over last 10 to 15 years have resulted in development of different combinations of 'Flocculator-Tube Settlers' configurations. Essentially, these lead to saving of space (land) and are especially suited to urban area where the land availability is a prime factor.

One such configuration is 'Tube-Clariflocculator' or 'Clarisettler' (Figure 8.24 & 8.25). This unit has similar configuration (flow path) that of a conventional clariflocculator, however, the clarification area is provided (packed) with tube settlers. The concentric central flocculation is provided with paddle type slow agitators. Tube modules are placed on radial trusses fixed on the outer wall. Since the surface loading rate of tube settlers is four times that of conventional clarification, the overall diameter of the clarisettler unit becomes one-half that of the clariflocculator. Clarisettler can be employed with centrally driven rake with fixed bridge or peripherally driven rake bridge. The design criteria for flocculation tank and tube clarification zone are as described earlier. These units are recommended to be adopted for plant capacities above 25 MLD. The sizing calculations for tube-clariflocculator or (clari-tube settler) are given in **Annexure 8.2**.

Figure 8.24: Clarisettler with Rotating Scrapper Bridge

Figure 8.25: Clarisettler with Central Rake

(b) **Tube Clarifier with Central Rake**

This is another configuration of flocculator – tube settlers (Figure 8.26 and 8.27). In this case the paddle flocculators and tube settlers are located in separate units with a common wall. The tube settlers are configured into a square tank. The sludge is removed by the central scraper rake. In case of multiple units, common wall construction makes these units very compact. This is alternative to hopper bottom tanks for large capacity plants. The design criteria for flocculation tank and tube clarification zone are as described earlier. These units are recommended to be adopted for plant capacities above 25–50 MLD. Normally, the diameter of a tube clarifier is 50%–60% that of a clariflocculator of same capacity.

Figure 8.26: External Flocculators and Tube Settling Tank with Central Rake

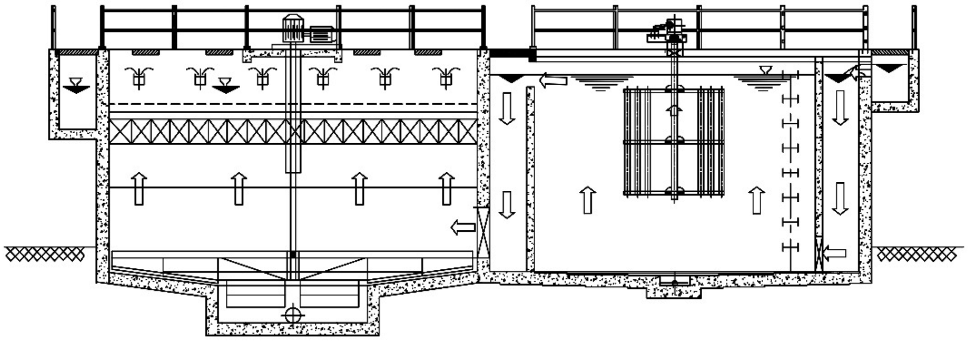


Figure 8.27: Schematic Flow Path

8.6.10 Ballasted Flocculation and Settling

This technology is relatively new to India, however, in the future, it is likely to be adopted to reduce the footprint (Figure 8.28). Ballasted flocculation and settling is a clarification process that includes the use of micro-sand as ballast to increase the specific gravity of the floc particles to improve their rate of settling. The micro-sand provides a surface to which the floc particles can attach in the presence of a high molecular weight polymer and serves as a weight to accelerate in settling. The sand-ballasted floc settles rapidly, which makes it possible to design clarifiers with high overflow rates and short retention times. Such designs make it possible to build systems that have footprints 5% to 20% of the area occupied by conventional clarification systems of similar capacity. This can result in significant cost savings, particularly if expensive excavation is required or the site is space limited. The ballast flocculation followed by tube settler with design surface loading rates of 25–50 m^3/m^2/hr. will substantially reduce settling area compared to conventional sedimentation basins.

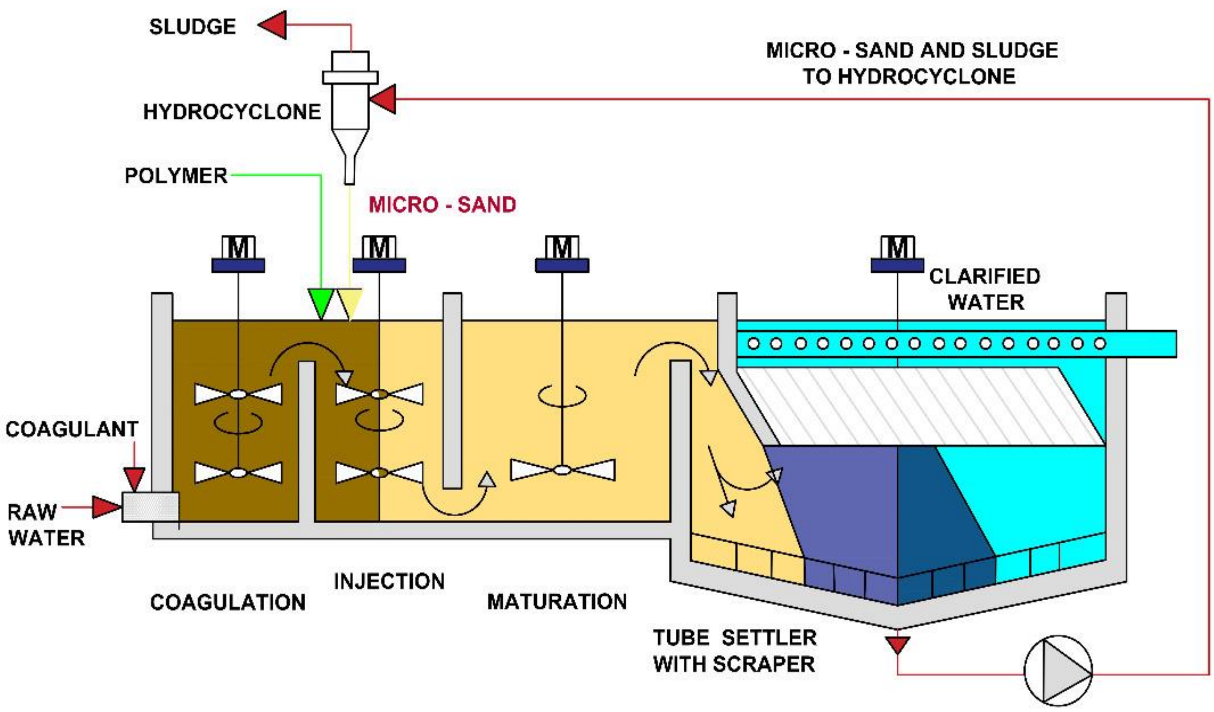


Figure 8.28: Ballast Flocculation System with Tube Settler

8.6.11 Dissolved Air Floatation (DAF)

This is a very rarely used technology in India. In DAF, the effects of gravity settling are offset by the buoyant forces of small air bubbles. These air bubbles are introduced to the flocculated water, where they attach to the floc particles and then float to the surface. In DAF, flocculation is designed to create a large number of smaller flocs (Figure 8.29 and 8.30).

For efficient floatation, flocculated particles must come in contact with large number of air bubbles. The attachment process is by adhesion of air bubbles to the surface of flocs, entrapment of bubbles under the floc and absorption of bubble into floc mass. At the surface, the bubble floc forms the sludge layer having consistency of 3% to 5% W/V. Floating solids are normally skimmed off by chain and flight skimmer.

The microbubbles of size 10 to 100 micron are introduced at the bottom by releasing air super saturated recycle water through proprietary nozzles or orifices. The recycled water is pumped at the rate of 5% to 10% and is injected by high pressure air from the saturation tanks. Air is supplied by the air compressor. The tanks can be of rectangular or circular shape.

8.6.11.1 Design Parameters

The DAF basin surface loading rates range from 10 $m^3/m^2/hr.$ to 12 $m^3/m^2/hr.$ (10 to 12 m/hr.). Recycle flow rate are 5% to 10% of plant flow rates. Dissolved air pressure is in the range of 4 kg/cm^2 to 6 kg/cm^2.

The efficiency of DAF depends on the proprietary nozzle to a great extent. These units are compact because of the high solids loading rate. However, they required pretreatment by coagulation and flocculation. This process is particularly effective for raw water with algae, colour and having low turbidity.

They have been used in RO feed water pretreatment for seawater to control the algal blooms.

However, DAF require high amount of electrical energy for the air compressors (energy intensive). The plant requires skilled manpower for operation and maintenance. Under the Indian techno-economic scenario they exhibit no special advantage over conventional tube/plate settling units.

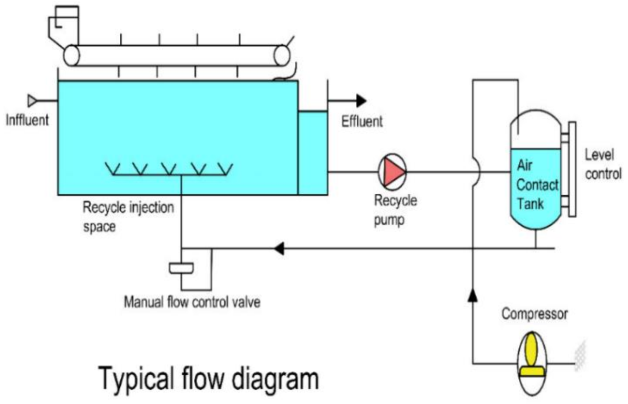


Figure 8.29: Typical Flow Diagram of DAF

Figure 8.30: WTP at T K Halli, Bengaluru

8.6.12 Unconventional Water Treatment Plants up to 5 MLD Capacity

In semi-urban areas where water treatment plant is required normally sufficient space is available. However, the skilled O&M staff is not available. In such cases WTP requiring minimum or no O&M will be preferred without compromising quality aspects.

In any WTP, sedimentation process is very important. Before inlet to filters, if the turbidity levels are maintained less than 15 units, then function of filtration will be more efficient. Hence, in the WTP construction inlet/outlet conditions are suitably modified to give better and assured results.

The sedimentation process in the plant is designed for plain sedimentation, with an overflow rate of 18 cum/sq. m/day. It is in two compartments and each is split into four settling units. All are hopper bottom units.

First hopper unit is redressed internally to conical shape for smooth flow of raw water splashed from jets, from bottom to top.

The first unit is jet flocculator in each compartment. The jet gets energy from the inlet pipe going about 5 m deep in the hopper. Water from jets is directed to move in circular motion due to the conical surface of the container and, at the same time, it rises in the unit passing through incrementally increasing area (Figure 8.31). At FSL, its rotational movement is just equivalent to that we normally in the flocculator. This unit is not expected to take out any turbidity, but it still separates out the inert solids.

Figure 8.31: Water Jets at the Bottom of the Hopper

Second hopper unit is having modified inlet. Water from Weir enters the launder which has several vertical pipes to direct the water to go down in hopper and is further directed to rise through similar vertical pipes in the end launder. Here the path of flow is purposely extended so that turbidity particles do not rise to the FSL and prefer settling in the unit. Direct flow between the launders is not possible.

Third hopper unit is also designed with down-take pipes through launders. However, the outgoing flow has options to move to fourth hopper through slant pipes in the separator walls of the hoppers. Still the turbidity particles are required to reach FSL in the fourth hopper unit where the outlet launders are provided with weir flow rate up to 80 to 100 cum/m/day, ensuring quiescent flow conditions. In such a case third and fourth hopper units become efficient to remove finer turbidity particles and it is observed to give outlet turbidity within the range of 10 to 15 NTU. For higher incoming turbidity in raw water, first two hopper units work very efficiently.

Efficiency of first unit with jet flocculator will be increased with alum dosing and/or pre-chlorination. Sufficient cleaning time for compartments is available as operation of one compartment can give similar efficient results. With chemical coagulation, in the first hopper of jet flocculator, capacity of WTP can be increased. It will increase design life of the WTP. As there are no moving parts in WTP, O&M is practically nil. Rapid sand gravity filters receive assured flow of raw settled water with optimum turbidity load; it results in the higher filter runs and higher filtration efficiency. In this type of plant, no mechanical controls are provided in the filters. As the filter run goes filtration resistance increases and water level reaches the maximum pre-decided level in the filter box. It indicates washing of filters. Outlets are provided with master pieces (Pipe in pipe) to operate at controlled rate

8.6.12.1 Design of Jet Flocculator

Entry of water is from the top, taking tapping from the launders. A tee is fitted instead of bend to make it vertical. Upper end of tee is open to air. At the bottom of the vertical pipe, a small drum is fitted to accommodate four jets at 90 degrees to adjoining jets. At the end of each jet pipe, a 90-degree bend is fitted with end reducing suitably. When flow starts, jets operate as outlets and splash to the walls of container. Flow automatically gets circular direction. The velocity at bottom is very high and as it moves upwards, it faces larger area, and the velocity is reduced. Hence, the initial portion acts as a flash mixer and thereafter, at higher levels, it slowly turns into flocculation velocity. The actual Gt values are tabulated after basic equation is arrived at.

8.6.12.2 Velocity Gradient Variation

The velocity gradient at the bottom of the flocculator is 189 s^{-1}, while at the top is 8.74 s^{-1}. For the present, the pattern of variation of G from bottom to top has been determined on the assumption that energy available at a point varies inversely with D^4, since the velocity varies inversely as square of the diameter of the flocculator, and energy is proportional to the square of the velocity. The values of GT are shown in Table 8.7

Table 8.7: Table showing values of GT

Height from the bottom apex	Height of part	Radius	Volume	Energy available	Local G
m	m	m	cum	m	g^{-1}
3.125	0.375	1.06	1.324	0.38913	189.1
3.5	0.5	1.2	2.262	0.24729	113.5
4	0.5	1.36	2.905	0.14496	77.6
4.5	0.5	1.52	3.629	0.0905	55.4
5	0.5	1.68	4.433	0.05938	40.9

Height from the bottom apex	Height of part	Radius	Volume	Energy available	Local G
5.5	0.5	1.84	5.318	0.04155	31.1
6	0.5	2	6.283	0.02863	24.2
6.5	0.5	2.16	7.329	0.02079	19.2
7	0.5	2.32	8.455	0.01546	15.5
7.5	0.5	2.48	9.661	0.01173	12.6
8	0.5	2.64	10.948	0.00906	10.5
8.5	0.5	2.8	12.315	0.00711	8.8
9	0.375	2.94	10.183	0.00566	8.7

8.7 Filtration

8.7.1 General

Filtration is the process of separating suspended and colloidal impurities from water by passing it through a porous medium. Filtration, without pretreatment will not effectively remove turbidity (e.g., silt and day), colour, microorganisms, precipitated hardness from softened waters, and precipitated iron and manganese from aerated waters. Removal of turbidity is essential not only from the requirement of aesthetic acceptability but also for efficient disinfection which is difficult in the presence of suspended and colloidal impurities that serve as hideouts for the microorganisms.

The granular medium filters include single-medium, dual-media and multi-media (usually tri-media) filters. Sand, anthracite coal, crushed coconut shell, or granular activated carbon have been used as filter media. The driving force to overcome the frictional resistance encountered by the flowing water can be either the force of gravity or applied pressure force. The filters are accordingly referred to as gravity filters and pressure filters. The flow rate control is constant rate and declining or variable rate filters. Lastly, dependent upon the flow rates, the filters are classified as slow or rapid sand filters.

Filtration of municipal water supplies normally is accomplished using:

(a) slow sand filters;
(b) rapid sand filters;
(c) high-rate filters.

8.7.2 Slow Sand Filters

These filters are not widely used since early 1990. Slow sand filters can provide a single step treatment for surface waters of low turbidity (<5 NTU) when land, labour and filter sand are readily available at low cost, coagulants and equipment are difficult to procure and skilled personnel to operate and maintain are not available locally.

When raw water turbidity is high, pretreatment such as storage, sedimentation, or primary filtration will be necessary to reduce it to within desirable limits. Coagulation and flocculation are not desirable as the gelatinous floc rapidly clog the sand media.

A slow sand filter consists of an open box about 3.0 m deep rectangular or circular in shape and made of concrete or masonry. The box contains a supernatant water layer, a bed of filter medium, an underdrainage system and a set of control valves and appurtenances.

The filter bed consists of natural sand with an effective size (E.S.) of 0.25 mm to 0.35 mm and uniformity coefficient (U.C.) of 3 to 5. For best efficiency, the thickness of filter bed should be not less than 0.4–0.5 m. As a layer of 10–20 mm, sand will be removed every time the filter is cleaned, a new filter should be provided with an initial sand depth of about 1.0 m. Re-sanding will then become necessary once in two to three years.

8.7.3 Rapid Sand Filters

8.7.3.1 Filtration Process

The rapid sand filter consists of a bed of a single or dual filter media supported on gravel overlying an underdrainage system (Figure 8.32). For rapid sand filtration to work efficiently, careful pretreatment of raw water is required to remove the suspended solids. Although not discussed in this section, high-rate filtration, deep bed filtration, multi-media bed filtration, use of activated carbon or anthracite coal as media are widely practised throughout the developed countries and should be considered based on site-specific conditions.

When water containing suspended matter is applied to the top of filter bed, suspended and colloidal solids are trapped in the granular medium matrix. The accumulation of suspended particles in the pores and on the surface of filter medium leads to the build-up of head loss as pore volume is reduced. The media offers greater resistance to the flow of water, simultaneously with the build-up of head loss across the filter. At a predetermined terminal value, the suspended solids removal efficiency of successive layers of filter medium is reduced as solids accumulate in the pore space and reach an ultimate value of solids concentration as defined by operating conditions. This results eventually in breakthrough of suspended solids and the filtrate quality deteriorates. Ideally, a filter run should be terminated when the head loss reaches a predetermined value simultaneously with the suspended solids in filtrate, attaining the preselected level of acceptable quality. The clogged filter bed is cleaned with air scour and water wash (hard wash). The direction of air and water wash is in the reverse direction that of filtration.

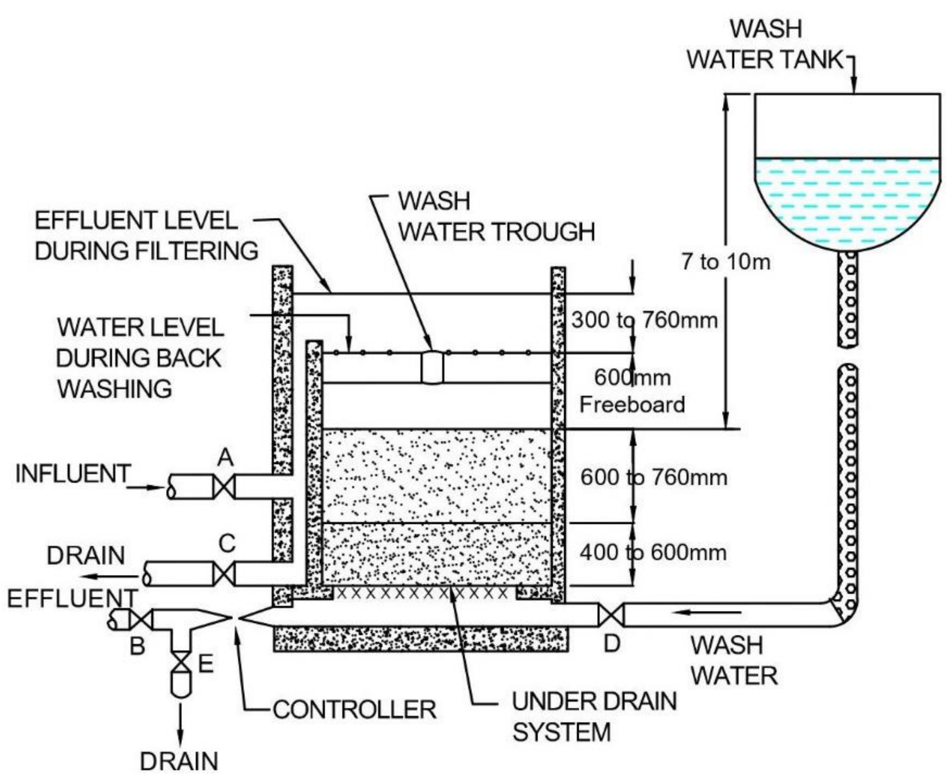


Figure 8.32: Granular Medium Gravity Filter

8.7.3.2 Rate of Filtration

The standard rate of filtration through a conventional rapid sand filter is usually 80 to 100 $l/m^2/min$. (4.8–6 $m^3/m^2/hr$.). Current practice is leaning towards higher rates by adopting dual media gravity filters or mono media deep bed filters. An average rate of filtration shall be 5.5 m/hr. or less. It shall be ensured that under overloading condition, when one bed is out of service, maximum rate of filtration shall be less than 6 $m^3/m^2/hr$. (The overloading check applies only if the number of beds exceeds four. A plant having four filter beds shall be designed to be operated for 22 hrs.). The inlet and the outlet control arrangements are designed to permit overload for emergent occasions or when one filter is under backwash operation. (This overloading factor should be 100% when two filter beds are provided, 50% in case of three beds, 33.33% in case of four beds and 20% when more than four beds are provided). Again, much higher filtration rates are practised in developed countries and should be considered based on site-specific conditions.

8.7.3.3 Capacity of a Filter Unit

The capacity of the rapid sand filters should be such that the number of units can take care of the total quantity of water to be filtered and is optimum to keep the filters working without undue overloading at any time. Table 8.8 shows the number of rapid sand filters for different plant capacities. The smaller the number of units, the fewer the appurtenances but the larger the wash water equipment that will be required. Thus, while designing large size filters, one must consider the rate at which wash water must be supplied and the hydraulic problems for securing uniform distribution of wash water due to the large area. A maximum area of 100 m^2 for a single unit is recommended for plants of greater than 100 MLD consisting of two halves each of 50 m^2 area. Also, for flexibility of operation a minimum of four units should be provided which could be reduced to two for smaller plants.

Table 8.8: Minimum Numbers of Rapid Sand Filters for Given Plant Capacity

Capacity in MLD (24 hr basis)	Number of beds*
1	1W (with one section)
2	2W (with one section)
3–8	2W
9–15	3W
15–30	4W
31–50	6W
51–80	8W
80–100	9W+1S

Note:
1. *For above 80 MLD capacity, it is recommended to provide a high-rate filtration, viz., dual media or mono media deep bed gravity filters, described later in the manual.*
2. **Filter beds with two sections separated by central gutter or gullet.*

8.7.3.4 Dimensions of Filter Unit

Layout of the plant, economy and convenience determine the relationship between the length and the breadth of the units. Where filters are located on both sides of a pipe gallery, the ratio of length to width of a filter box (Two sections with central gutter) shall be preferably between 1 to 1.66. The bed shall have an overall depth from 2.6 m to 3.75 m depending on type of filter excluding a free board of 0.5 m.

The filter shell (boxes) shall be in reinforced concrete to ensure a watertight structure. Except in locations where seasonal extremes of temperature are prevalent (extremely heavy rains or significant snowfall areas), it is not necessary to provide a roofing over the filters. The filters shall be preferably covered with roof in a building. The filters are recommended to be covered if they lie in the vicinity of thermal power plants or carbon product manufacturing units having chimneys to avoid water getting covered by the soot.

8.7.3.5 Filter Sand

Filter sand is defined in terms of effective size and uniformity coefficient. Effective size is the sieve size in millimetres that permits 10% (D_{10}) by weight to pass. Uniformity in size is specified by the uniformity coefficient which is the ratio between the sieve size that will pass 60% (D_{60}) by weight and the effective size (D_{60}/D_{10})

Shape, size, and quality of filter sand shall satisfy the following norms:

(i) Sand shall be of hard and resistant quartz or quartzite and free of clay, fine particles, soft grains, and dirt of every description.
(ii) Effective size shall be in average between 0.45 to 0.70 mm. The effective size of sand can range from 0.8 to 1.5 mm for deep sand filters (like mono media deep sand filter).
(iii) Uniformity coefficient shall not be more than 1.7 nor less than 1.3.
(iv) Ignition loss should not exceed 0.7% by weight.
(v) Soluble fraction in hydrochloric acid shall not exceed 5.0% by weight.
(vi) Silica content should be not less than 90%.
(vii) Specific gravity shall be in the range between 2.55 to 2.65.
(viii) Wearing loss shall not exceed 3%.

8.7.3.6 Depth of Sand

Usually, the sand layer has a depth of 0.60 to 0.75 m. The standing depth of water over filter media top varies between 1 and 2.5 m depending on the filter control arrangement. The free board above the water level should be at least 0.5 m so that when air binding problems are encountered, it will facilitate the additional levels of 0.15 to 0.30 m of water being provided to overcome the trouble.

(i) Procurement of Filter Sand

The natural filter sand is procured from various sources like Banda in Uttar Pradesh, Paonta Sahib in Himachal Pradesh, Jaipur in Rajasthan, Godra and Vejalpur in Gujarat, Bhandara in Maharashtra, Belgaum in Karnataka, and other sites.

Further the IS: 8419 (Part 1) (1977, Reaffirmed 2010) entitled ‘requirement for filtration equipment-Filtration Media Sand and Gravel’ may be referred to for details.

(ii) Filter Bottoms

The IS: 8419 (Part 2) (1984, Reaffirmed 1996) entitled ‘requirement of rapid sand gravity filtration equipment – under drainage system’ maybe referred to for details.

The underdrainage system of the filter is intended to collect the filtered water and to distribute the wash water in such a fashion that all portions of the bed may perform nearly the same amount of work, and when washed, receive nearly the same amount of cleaning. Since the rate of wash is several times higher than the rate of filtration, the former is the governing factor in the hydraulic design of filters which are cleaned by backwashing. The design of underdrain is most critical for efficient backwash operation and functioning of the filters. The material needs to be sturdy. Once buried under the gravel and sand, the underdrain has no access.

The most common type of underdrain is a central manifold with laterals either perforated on the bottom or having umbrella type strainers on top (Figure 8.33 & 8.34). Other types such as wheeler bottom, block underdrains, false bottom with nozzles, dual-parallel blocks, etc.

Manifold and Laterals

The following values may be used in design of an underdrain system consisting of central manifold and laterals.

The perforations vary from 5 to 12 mm in diameter and should be staggered at a slight angle from the vertical axes of the pipe. Spacing of perforations along the laterals may vary from 80 mm for perforations of 5 mm to 200 mm for perforations of 12 mm.

The ratio of the total area of perforations to the underdrain system to the total cross-sectional area of lateral should not exceed 0.5 for perforations of 12 mm and should decrease to 0.25 for perforations of 5 mm.

Ratio of total area of perforations to the entire filter area may be about 0.3% to 0.35%. The ratio of length to diameter of the lateral should not exceed 60. The spacing of laterals closely approximates the spacing of orifices and shall be 300 mm.

The cross-sectional area of the manifold should be preferably 1.5 to 2 times the total area of the laterals to minimise frictional losses and to give the best distribution. It is useful to check the design for uniformity of distribution of wash water in laterals of the underdrains.

Figure 8.33: Manifold-Laterals Assembly

Figure 8.34: "Concrete Tee" as a connector block to Manifold Pit

Nozzle Systems on Header Laterals

Nozzle geometries are typically either cylindrical, conical or umbrella top with long hollow stem. Injection molded nozzles utilize v-shaped, vertical slots that prevent fouling. Some believe that the older-style conical nozzles have an angular backwashing benefit; however, cylindrical nozzles are manufactured with reinforcing screen ribs to ensure added longevity. The size of a nozzle is directly proportional to flow rate. Hence, nozzle density in the underdrain system factors into the flow rate per nozzle. For example, for the same lpm per square metre backwash rate, designing nozzles on 200 mm centers each way requires 78% more flow per nozzle than designing nozzles on 150 mm centers each way. It is recommended that nozzles to be located as close as practicable to ensure effective media cleaning through more points of energy input and to potentially eliminate gravel and other types of problematic packing layers. The filter floor should consist of a series of lateral pipes each connecting to a central channel or manifold and fitted with nozzles. The pipes should have holes on the top to receive 'nozzle plates' in which the nozzles are screwed. The portions of the pipes bridging the channel, formed in the floor of the filter tank should have slots on the undersides. All the pipes should be grouted to the floor and embedded in concrete with suitable reinforcement, after which the hole should be screwed to a smooth surface flush with the upper faces of the nozzle plates. The nozzles should then be screwed into the position.

Mounting methods of non-lateral nozzles depend on the underdrain type. For monolithic, reinforced concrete underdrains, nozzles thread into concrete sleeves that are cast into the floor. Non-tapered threads require an ‘O-ring’ to lock the replacement nozzle into place.

Nozzles for pipe header-laterals may be mounted directly using a pipe saddle and expanding ring. For header-lateral systems embedded in concrete, pipe sleeves with temporary caps may be glued onto PVC or HDPE pipe. After the concrete has been poured and cured, the caps may be removed and nozzles installed. Alternatively, a bottom-threaded concrete sleeve with temporary cap may be threaded into an expanding ring with pipe saddle.

Materials used for nozzle products are of polypropylene, glass-reinforced polypropylene, un-plasticized PVC, HDPE, cast iron, stainless steel, brass and combinations of materials such as a stainless-steel screen section with polypropylene stem etc.

Polypropylene is appropriate for most water treatment applications. However, for example, if water temperatures exceed 60°C (or 140°F) or if specific chemical resistance is necessary, other materials of construction should be selected.

Nozzle screen slot widths vary from 0.1 mm for very small ion exchange resin up to 5.0 mm for underdrains with a deep gravel packing layer. Most filtration applications tend to result in screen slot widths between 0.2 mm and 0.5 mm. Nozzle cages with color-coding found helpful for the ease of identification of the slot width.

These dimensions are typically chosen to be between 50 to 70% of the effective size of the smallest media in order to retain the granules as well as to not foul with fines or biology. For example, the effective size of typical sand filter media of 0.5 mm; a 0.3 mm-slotted screen should be chosen. An adsorber may have 1.2 mm effective size media; thus, a 0.8 mm-slotted screen would be appropriate.

Appropriate amounts of pressure drop or head loss through the nozzle at operating flow rate ranges is important to ensure proper distribution during backwash. Enough pressure drop through the underdrain is required to overcome the head loss of the dirty media bed. Otherwise, uneven distribution occurs, allowing unwanted channeling and poor media cleaning, which reduces subsequent filter run times and creates maintenance issues due to the left-over solids. For typical sand/anthracite/Granulated Activated Carbon applications, at least 500 to 600 mm of Water Column is recommended at the design high backwash rate.

There are two main design elements that determine pressure drop through a nozzle at a given flow rate. First is screen construction. A choice may be required between more than one appropriate width. For the same number of slots, smaller width slots impart more pressure drop than larger. Utilizing the same width slot, fewer slots in a screen will create more pressure drop at the same flow rate.

The second design element is the bore size of the thread section and the tailpipe, if used. In increasing pressure drop for a given flow rate, bore diameters of 21mm, 16mm, 16mm with a 13mm restriction in the thread section, and also 13mm may be utilized. To add even more pressure drop, a closed stem with holes may be used.

Proper nozzle characteristics ensure media retention, negate fouling, and produce excellent distribution, which all will deliver optimal filter performance.

Strainer Nozzles with False Bottom and Plenum Chamber

The nozzles with stems are embedded in to a false bottom concrete slab. The concrete slab is normally 100 to 125 mm thick. The slab is supported with concrete stub columns. The portion below the slab is called as "Plenum Chamber ". For maintenance purpose the height of the plenum chamber needs to be minimum 900 mm. A manhole is provided on the side wall of the filter to enter into plenum chamber.

During casting of the false bottom concrete slab, the nozzle sleeves are embedded in it. The spacing of the nozzles in both direction needs to be uniform. Later the nozzle with stem is screwed in to sleeves. There are various types of nozzles with different area of opening. Normally the nozzle with 0.2 mm slit width is adopted. The manufacturer has to furnish the data on area of opening and headloss that occurs during the filtration cycle and backwash cycle. The system is suitable for air, air/water and water wash. However, it is cumbersome to construct. The false bottom slab and plenum chambers increases the civil cost. The false bottom slab needs to be designed for the tension on both sides.

Dual-parallel underdrain blocks (Flat bottom flumes)

This system is provides an improved distribution of backwash water to the filter media through the use of dual compensating laterals. Backwash water enters the primary laterals and then passes through the control orifices into the secondary laterals. Backwash air is properly distributed by careful design of upper control orifices between the primary and secondary laterals, providing an even air distribution. Media retention plates maximize the available filter tank depth. The media retention plates prohibit media pass through and provide proper flow characteristics.

The blocks size ranges from 200 mm x 200 mm to 300 mm x 300 mm typically. They are made of Rigid PVC or other sturdy plastics. The block underdrain system can permit separate air scour, water backwash, simultaneous air and water backwash modes. The inherent design of underdrain eliminates uneven distribution of air and water even in large filter beds. Generally, the blocks are designed for filtration rates 25 m3/m2/hr. The water back wash upflow range is 20-90 m3/m2/hr. For simultaneous air-water wash the rate of flows can be designed as 70 m3/m2/hr. and 15 m3/m2/hr. respectively.

The filter underdrain system, when installed is designed for a net internal loading during backwash of either 2.93 m H_2O or 200% of the maximum pressure at maximum backwash rates, whichever is higher. The filter underdrain system is designed to withstand a net downward loading of not less than 6.84 m H_2O.

The block underdrain system is extremely appropriate for their ability to operate the backwash in various modes, the media retainer plate eliminates the requirement for supporting gravel for filter. Like nozzle with plenum chamber provision of the additional filter depth/height is avoided. Hence additional depth is available for various media configurations. The underdrain blocks are suitable to be adopted with high rate or very high-rate filtration viz. Mono media deep bed filters described later.

8.7.3.7 Filter Gravel

Gravel is placed between the sand and the underdrain system to prevent sand from entering the underdrains and to aid uniform distribution of wash water. The gravel should accomplish both purposes without being displaced by the rising wash water. Sizes of gravel vary from 50 mm at the bottom to 2 mm at the top preferably with a 0.45 m–0.60 m depth. The faster the rate of application of water, the larger the gravel size required. Reference may be made to IS: 8419 Part (1)-1977 (Reaffirmed 2001) for filter gravel. The dual-parallel block underdrain does not require the gravel support.

Suggested gravel configuration for manifold-lateral underdrain system are given in Table 8.9.

Table 8.9: Conventional Rapid Sand Gravity Filters

Gravel Size, mm	Gravel Depth, mm
2 – 5 (3 – 6)	10
5 – 10 (6 -12)	10
10 - 20	20
20 -50	20

8.7.3.8 Wash Water Collection Troughs/ Gutters

Wash water collection troughs collect the dirty backwash water during the backwash and discharge in to a central/side gutter for further disposal out of filter bed. The horizontal travel of backwashed water over the surface of the filter is kept between 0.6 m to 1.0 m before reaching the gutter. Usually, the top of the cross-troughs is 0.60 m to 0.70 m above top of the media. The bottom of the troughs (soffit) should clear the top of the expanded sand by 50 mm or more. The cross-troughs should be large enough to carry all the water delivered to it with at least 50 mm of freefall between the surface of the water flowing in the gutter and the upper edge of the gutter (freefall). The discharge capacity Q in m^3/s may be computed from the formula:

$Q = 1.376\ bh^{3/2}$

Where b is the width of the trough in m and h is the water depth in m and Q is in m^3/sec.

The cross-troughs discharge the dirty water into the central gutter. The crest of cross-troughs shall be 50 mm to 75 mm below the top of the central gutter. In no case the central gutter shall have overhang above or into the filter media. Generally, the height of the central gutter varies from 1.80 m to 2.1 m depending on the filter media configuration.

8.7.3.9 Air Scour and High-Rate Backwash

Normally, for effective cleaning of filter beds, air scour followed by water (hard) wash is adopted. The piping for the same can be separate or combined. The air sour is recommended at the rate of 36 to 45 m/hr. (of filter area) at 0.35 kg/sq. cm for duration of 5 min. The water wash is recommended at the rate of 24 to 40 m/hr. (of bed area). The effective head for wash water tank (overhead) is 8.50 m above the filter underdrain system. The duration of water wash is 10 min. In a filter bed with two sections with a central gutter (Figure 8.35), it is the designer’s choice whether to back wash both sections separately or simultaneously.

Figure 8.35: Cross-Troughs and Central gutter

8.7.3.10 Mechanism of Flow Controller

Water flowing through the venturi section produces different pressures at the main and throat, due to the difference of velocities at these points. Since connections from the main and throat lead to the upper and lower halves, respectively, of the diaphragm chamber, these differential pressures are reflected directly on the piston, moving it a certain distance, dependent on the difference between the pressures being exerted. Since downward pressure on the top of the piston is greater than upward pressure from below, a downward pull that is balanced by the counterweight on the long arm of the beam is transmitted to the scale beam. This balance of counterweight and piston load regulates the valve opening and limits the maximum rate of discharge through the controller.

In filter operation, the controller, by virtue of its throttling action, uses up all the head due to the difference in raw and filtered water which is not required to overcome friction due to sand, piping, velocity head, etc., and as the loss of head through the sand increases, the head consumed by the controller diminishes by a corresponding amount. During the entire operation, therefore, the rate of filtration remains practically constant.

However, it must be emphasised that rate of flow controllers requires proper operation and maintenance to ensure that filtration is done at a constant rate. These devices are getting progressively omitted and getting replaced with declining rate of filtration or constant rate filtration with influent splitting weirs systems.

(a) Filter Gauges

Filter gauges are essential to the operation of the modern filter plant in order to measure accurately the rate of flow through each filter box and to determine the loss of head occurring at any given time during the filter run. Gauges are available in various combinations of rate of flow and loss of head, both indicating and recording or as single recording or indicating units.

These gauges use the float and mercury principle for the conversion of differential pressure into measurement of loss of head or rate of flow. The primary pressure differential-producing device required for the rate gauge usually is the venturi section of the effluent rate controller, connections to the high- and low-pressure sides of the gauge cylinder being made to the main and throat sections of the controller. The differential pressure for the gauge is the difference between the water level in the filter box and the pressure head in the effluent pipe, pressure connections being led from these sources to the high- and low-pressure gauge cylinder taps.

Piezometers (manometers) shown in Figure 8.36. The head loss can also be measured using mechanical equipment as shown in Figure 8.37. These can also be used for the purpose, though they suffer from the disadvantage that they have to be cleaned from time to time. They are simpler, more positive, and much less expensive than the conventional types of instruments.

Figure 8.36: Head Loss Indicator (Manometer)

Figure 8.37: Rate of Flow and Head Loss Indicator (Mechanical)

(b) **Pipe Gallery**

The influent, effluent, wash and waste water pipes together with rate controllers, appurtenances and pure water outlet chamber are placed in the pipe gallery. Galleries should be well designed to provide adequate space, ventilation drainage and easy accessibility to all pipe-work and other fittings. When filter beds are arranged in a single row, the pipe gallery is on one side of beds. When the beds are arranged in two rows, the pipe gallery is located in between. Pure or filtered water conduit or Channel is located in the pipe gallery or by the side of pipe gallery. The top slab of pure water channel is used as a lower-level walkway.

Upper-level walkway is provided at the top of filter box. The width of the walkway is normally same as that of pipe gallery. Minimum distance between upper-level walkway and lower-level walkway shall be 2.25 to 2.50m. In a manually operated plant, Upper-level walkway houses valve operating headstocks, wheels, and gear boxes. In automated plants, it houses Filter Operating Consoles. Pipe gallery and pure water channel is housed in building with headroom over upper-level walkway as 3.5m to 4.0m. Adequate lighting and ventilation shall be provided to the pipe gallery.

Normally filter annex building hosing air blowers, back wash tank filling pumps etc. is adjacent to the Filter House Structure. In that case interconnecting staircases or walkways need to be provided to facilitate operator movement.

8.7.4 Rapid Gravity Dual Media Filters

The rapid gravity dual media filters are filters containing two media, normally coal and sand, and water is applied in downward direction under gravity.

8.7.4.1 Constructional Features

The enclosure tank containing filter media is usually a rectangular box, made of concrete. The plan area of these filters may range between 40 m^2 and 200 m^2 with depths between 3.0 m and 3.75 m. The filter media is supported on gravel laid over top of the underdrainage system. In addition to the underdrainage system used for collecting filtered water and distributing the backwash water, the tanks have troughs spanning across the length or width of filter for distribution of water to be filtered and for collection of wash water. The troughs remain submerged during filtration and their top edge is normally kept 600 mm above the filter medium to prevent loss of medium during backwash and to minimise the amount of dirty water left above the filter bed at the end of the wash.

The filters are commonly arranged in rows on one or both sides of a pipe gallery. The gallery houses the influent, effluent, wash water supply, wash water drainage piping, valves and other appurtenances including rate of flow controller. The pressure gauges to indicate head loss and venturi metre or rate of flow recorder are also located above and/or below the gallery floor.

8.7.4.2 Filtration Media

With a view to maintain coarse to fine gradation of pore sizes and pore volume with increasing depth of filter bed, two media of different density and sizes are chosen. The top layer consists of a lower density material like coal having larger particle size over a layer of higher density material like silica sand having smaller diameter particles. Since in India anthracite coal is not easily available, the coarse medium may consist of high-grade bituminous coal or crushed coconut shell which have been recommended for use after laboratory and field trials. (The bituminous coal variety available in India is of softer quality. The experience has shown that the coal particles get disintegrated in to powder form and cannot withstand the vigorous backwashing of media). The effective size (E.S.) of coal (specific gravity 1.4) is usually 1 mm (0.85–1.6 mm range) with uniformity coefficient (U. C.) of 1.3 to 1.5. Depths of 0.3 to 0.4 m have been reported to be satisfactory without excessive head loss build up and these depths can flocculate particles besides removing large flocculated impurities. The finer media layer usually consists of 0.3–0.4 m thick silica sand (specific gravity 2.65) with effective size of around 0.5 mm (0.45 to 0.6 mm range) and uniformity coefficient of 1.3 to 1.5.

The basic principle in designing the dual media bed is to have coal as coarse as is consistent with solids removal to prevent surface blinding but to have the sand as fine as possible to provide maximum solids removal subject to the constraint that the finer sand should not be present in the upper layers after backwashing in appreciable quantity.

8.7.4.3 Filtration Rates and Filtrate Quality

Dual media and multi-media filters have been successfully operated at rates of filtration ranging from 10 to 20 m^3/m^2/hr., with acceptable filtrate quality. Filtrate turbidities are generally less than 1 NTU and coliform removal is around 95%.

It may be recommended to operate dual media filters at higher rates 10 to 12 m^3/m^2/hr. to achieve filter run of 24 hr or more.

The backwash rates of 42 to 54 m^3/m^2/hr. (700–900 l/m/m^2) have been recommended to clean the filters. The higher back wash rates are essential as the media need to be expanded up to 30%. At the closure of back wash, the hydraulic grading of the particles takes place. Coarse and lighter media stays on top. Fine and dense media settles to the bottom. It is extremely important to ensure expansion of media during backwashing to ensure the media configuration. Like rapid sand gravity filters, air scour is also employed prior to water wash (Figure 8.38, 8.39 & 8.40).

Figure 8.38: Pilot Plant for Dual Media Filters

Figure 8.39: Checking Media Expansion During Backwash using Sand Expansion Stick

Figure 8.40: Dual Media Filter Bed during Internal Erection and Media Loading

8.7.5 Multi-Media Filters

The multi-media filters normally contain three media such as anthracite coal, silica sand and garnet sand with specific gravities being around 1.4, 2.65, and 4.2. The size of media may vary from 2 mm at the top to 0. 15 mm at the bottom. A typical tri-medium filter may contain 0.45 m of coal with an effective size of 1.4 mm, followed by 0.23 m of silica sand of effective size of 0.5 mm and 0.08 m of garnet sand having an effective size of 0.3 mm. Media of polystyrene, anthracite, crushed flint sand, garnet and magnetite whose specific gravities are 1.04, 1.40, 2.65, 3.83 and 4.90 respectively are being tried. However, it should be noted that multi-media filters are used infrequently as the fine media presents a problem in units without gravel, and also that of inter-mixing.

8.7.6 Mono Media Deep Bed Gravity Filters

Mono media deep bed gravity filters (MMDB filters) or uniform media filters are shown in Figure 8.41 and Figure 8.42.

Figure 8.41: MMDB Filters

Figure 8.42: 'Coarse' and Uniform OCW, Media is used in MMDB filters

In these filters, the medium is normally sand. The recommended rate of filtration is same as adopted for dual media filters. The grain size of sand is larger and is usually 0.80 mm to 1.5 mm (E.S.) or more. To compensate for the greater porosity (larger pore sizes), the depth of the media is required to be increased up to 1.0 m to 2.0 m. The uniformity coefficient (U.C.) has to be low and is 1.25 to 1.30. An attempt is made to provide as uniform a medium as possible.

Fluidisation of these types of beds is difficult during backwashing as they require very high flow rate. Therefore, these beds are backwashed with combined (concurrent) air-water scour, and low-rate water wash. The sequence of washing is that first high-rate air scour is employed for 5 min. (36 m/hr. to 54 m/hr), followed by combined air-water wash for 10 min. (air scour rate 36 to 54 m/hr and wash water rate of 15–25 m/hr.) and then by high-rate water wash for 5 min. (36 m/hr.). These filters require underdrain system of strainer nozzles with plenum chamber or underdrain blocks as described earlier. The configuration of filter is same as conventional rapid sand filter with central gutter and cross-troughs.
To carry out the dirt effectively to central or side gullet, sometimes during backwashing, horizontal flow of water is introduced at the top of the medium (cross flow). This is normally done using clarified water. The filtration rates and the filtrate quality are comparable with dual or multi-media filters. The rate of filtration is comparable with ‘coarse to fine’ media filters. These techniques were essentially developed in Europe.

8.7.7 Pressure Filters

In pressure filters, the removal mechanism is same as that of the rapid sand filtration. The filtration rate is high (6,000–15,000 $l/m^2/hr.$) and water is passed through the filter under pressure (3–7 kg/cm^2) through a cylindrical tank, usually made of steel or cast iron, wherein the underdrain, gravel and sand are placed. They are compact and can be prefabricated and moved to site. Economy is possible in certain cases by avoiding double pumping. Pretreatment is essential. The tank axis may be either vertical or horizontal.

Pressure filters can be used for small capacity plants for removal of iron, manganese, etc. They can also be used for swimming pool water.

In vertical pressure filters, the sand media is confined at the bottom and the pressurised water is applied uniformly at top surface of the media, so that the churning of the media is avoided.

Pressure filters suffer from the following disadvantages:

a. The water under filtration and the sand bed are out of sight and it is not possible to observe the effectiveness of the back wash or the degree of agitation during washing process.
b. It is difficult to inspect, clean and replace the sand, gravel, and underdrains of pressure filters.
c. Because the water is under pressure at the delivery end, on occasions when the pressure on the discharge main is released suddenly, the entire sand bed might be disturbed violently with disastrous results to the filter effluent.
   In view of these disadvantages, pressure filters are not recommended for community water supplies, particularly for large ones. They may be used for industrial needs and swimming pools.

8.7.8 Additional Modifications of Conventional Rapid Gravity Filters

8.7.8.1 Constant Rate Filtration by Influent Flow Splitting

In conventional rapid sand filters, constant rate of flow is maintained by installing a rate of flow controllers on the effluent line. These mechanical rates of flow controller devices are quite complex and hence, are difficult to maintain. They are high in initial and maintenance costs. Alternative systems have been proposed which are relatively simple to build, operate and maintain. These devices are progressively being discarded in practice. These are being replaced with constant rate hydraulic flow control arrangement.

One of the simplest methods is rate control by influent flow splitting, which is depicted in Figure 8.43. The filter influent is divided equally among all the operating filters in parallel by means of a weir at each filter inlet. The size of the filter influent conduit/channel is kept relatively large so that the head loss is not significant, and the water level does not vary significantly along the length of the conduit/channel. This helps in maintaining nearly same head on each of the weir and filter influent is equally split among all the operating filters. The filtration rate is controlled jointly for all the filter units by the inflow feeding rate. At the beginning of filter run, when a backwashed filter is put into service, the level of water in that filter is minimum. As the filtration proceeds and head loss builds up, the water level rises in the filter till it reaches the maximum permissible level above the filter bed, which may be, for example, equal to the level of influent weir. The filter is then taken out of service for backwashing.

The advantages of this system include elimination of rate controllers and slow and smooth changes in rates due to gradual rise and fall of water level above filter bed with less harmful effects on filtrate quality in comparison to filters having rate of flow controllers. To completely eliminate the possibility of negative head in the filter and to avoid accidental dewatering on media, the effluent control weir must be located above filter media as depicted in the Figure 8.44, 8.45, 8.46 & 8.47. In such case, the crest of filter outlet weir is provided 0.15 m to 0.20 m above the top of media.

The only disadvantage of the influent flow splitting system is the additional depth of the filter box which is 0.5 m to 1 m more than in conventional filters. The total depth of water is from 3.50 m to 3.75 m.

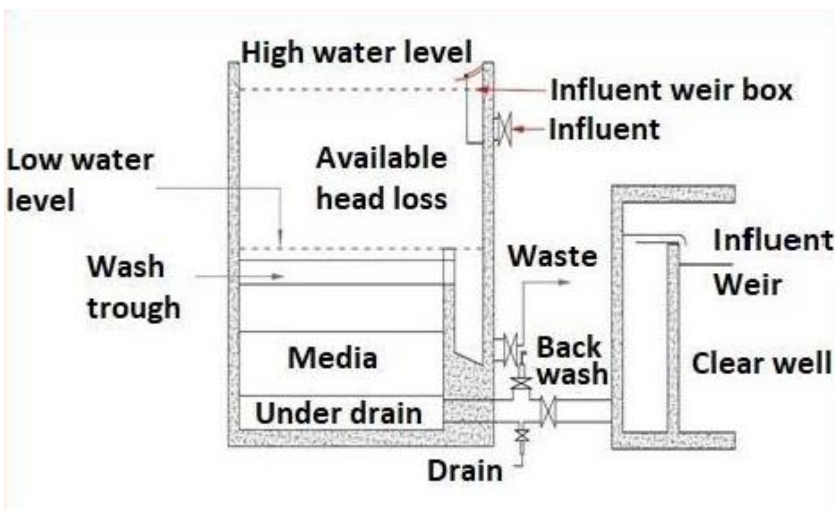


Figure 8.43: Gravity Filter Arrangements for Rate Control

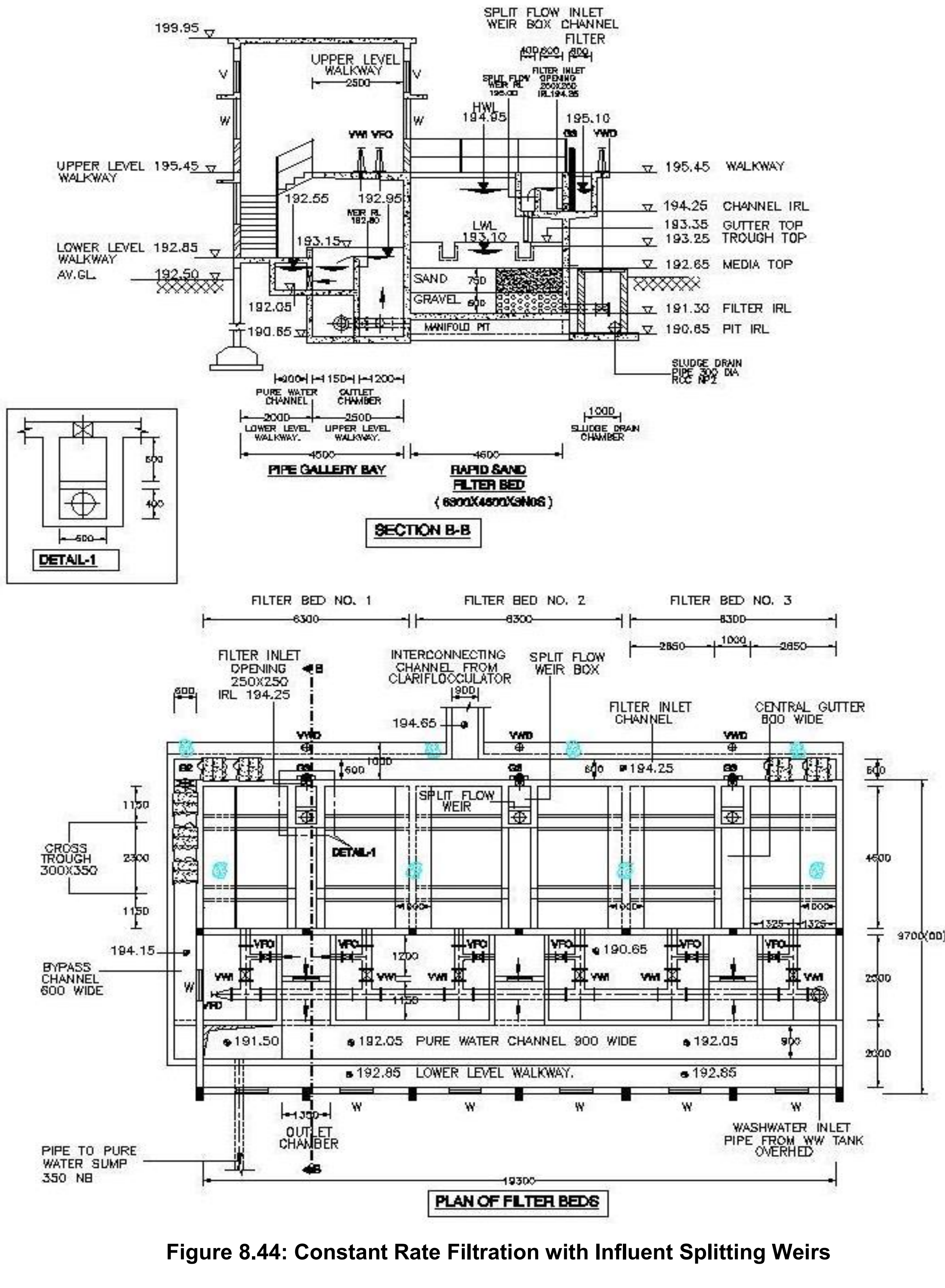


Figure 8.44: Constant Rate Filtration with Influent Splitting Weirs

Figure 8.45: Filter Inlet Channel and Inlet Weir with Chambers

Figure 8.46: Close-up of Splitter Weir and Chamber

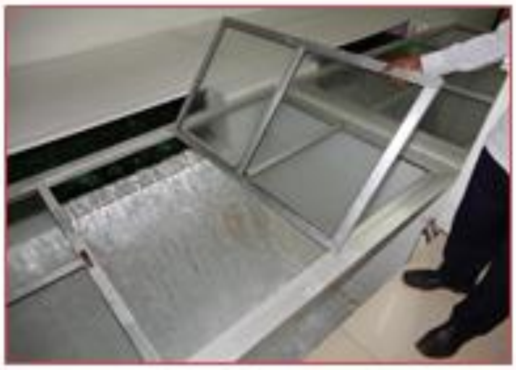


Figure 8.47: Filter outlet chamber with weir, Weir Crest Located above Top of the Media

8.7.8.2 Declining Rate Filtration

This is also referred to as variable declining rate filtration. In this system, the filter influent enters below the low water level of the filters and not above as in the case of influent flow splitting system described in sub section 8.7.8. A relatively large influent header (pipe or channel) serves all the filters, and a relatively large influent valve is used for each individual filter. This results in relatively small head losses in the influent header and influent valve and water level is essentially the same in all operating filters at all times. The essential features for variable declining rate filtration system are shown in Figure 8.48. No rate of flow controllers is used in this system also.

During the course of filtration by a series of filters being served by a common header, as the filters get clogged, the flow through the dirtiest filters decreases most rapidly. This causes redistribution of load among all of the filters, increasing the water level providing the additional head needed by the cleaner filters for handling additional flow. Therefore, the capacity lost by the dirtier filters is picked up by the cleaner filters through an increase in filtration rate and may result in deterioration of the filtered water quality.

The advantages of this system include significantly better filtrate quality than obtained with constant rate filtration, and less available head needed than that required for constant rate operation. However, recently, a new concern has emerged for these types of filters. After the backwash of a bed, that bed tends to discharge more water. This is associated with higher initial turbidity of the filtrate. It is found that cryptosporidium and Giardia cysts and oocysts escape in the filtered water (initial filtrate). Therefore, many advanced countries, including the United States, have discouraged its use in public water systems.

Another type of declining rate filtration is called 'controlled-head' operation. In this type of filters, the filter effluent lines are connected to a common header. A fixed orifice is built into the effluent piping for each filter so that no filter, after washing, will take an undue share of the flow. The filtered water header pressure may be regulated by a throttle valve which discharges to a filtered water reservoir. Costly rate controllers are replaced with fixed orifices and, therefore, would make the units economical particularly in large water works involving batteries of filters. The quality of water produced by the declining rate filters and filters controlled by conventional rate controllers are reported to be almost the same. For equal durations of filter runs, the total output per day from a declining rate filter is higher than that in the conventional one. In a group of filters operating at an avenge rate of 6 m^3/m^2/hr., fixed orifice will be so designed that a recently cleaned filter will begin operation at 9 m^3/m^2/hr., while the filter next in line for backwashing will have slowed down to about 3 m^3/m^2/hr. Usually, the depths of filter boxes for declining rate filters are more than those for the conventional ones. These would permit longer filter runs and consequent reduced wash water requirements. The possibility of 'break through' resulting in increased concentration of suspended solids in the effluent in filters with rate controllers is avoided in this system.

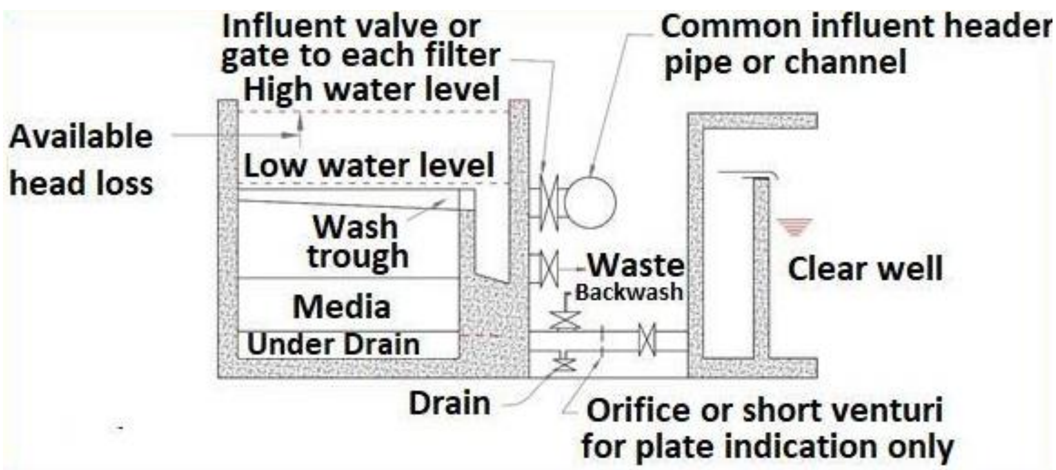


Figure 8.48: Gravity Filter Arranged for Variable Declining Rate of Filtration

8.7.8.3 Upflow Filters

Upflow filters are not practised nowadays in India. In upflow filtration, the water is passed under pressure in an upward direction through the coarser medium followed by finer medium. Thus, larger size suspended solid particles are first retained in the larger interstices of the lower part of bed and as the water percolates upwards, it receives a progressive polishing until it emerges in a fully filtered condition at top of the filter bed. Thus, the entire depth of media is made effective in removal of suspended solids and as a result, low head loss and longer filter runs could be expected. Besides, many other advantages are claimed for upflow filtration such as elimination of the rate controller and absence of negative head. Unfiltered water can be used for washing filter since the first few minutes of flow through the filter after washing has to be necessarily run to waste. Filter depths as low as 0.6 m and as high as 1.5 m have been successfully used. Although wash water rate and consumption are greater per wash cycle than the conventional filter, wash water used as a percentage of finished water is much less because of low loss of head and long filter runs. But initially, compressed air scouring is desirable to dislodge the impurities collected in the lower portions of the bed. The only disadvantage is fluidisation of the top fine layers of the sand bed which results in the deterioration of the filtrate quality. Complete bed fluidisation occurs when the head loss equals the depth of bed. Control of head loss is much more significant than the upward velocity through the filter. It is desirable that the hydraulic gradient through the upflow sand bed is restricted to 0.6.

8.7.8.4 Automatic Valve-less Gravity Filters

This type of filter is used in typical industrial installations. These filters operate without butterfly valves, pilot mechanisms, rate controllers, gauges, and air compressors. They have two compartments, the filtering section and wash water storage compartment. As the incoming water is admitted to the filter, a head gets built upon the top of the sand and causes the water level to rise in the backwash pipe. When the water level reaches the top of the loop, usually designed with a 2 m differential, syphon action is started, and backwashing begins at the required rate of 30 to 42 $m^3/m^2/$ hr. Wash water flows from the storage tank up through the sand bed and is discharged through the backwash pipe. A syphon breaker ends the wash cycle. The filter washes itself automatically, at the proper time at a given loss of head, without any mechanical instrument or operating tables. There is no maintenance from a mechanical standpoint of view. These filters are useful for low turbidity waters and for small installations.

8.8 Disposal and Recycling of Filter Back Wash Water

The NRW values in water supply system mentioned in Chapter 2, Part A of this manual is 15%, of which 3% NRW is considered as water lost in WTP. In order to minimise this loss, the recycling of backwash water is adopted (Figure 8.49).

Earlier, this backwash water was disposed of into natural water bodies. Recently, due to acute water shortages, scarcity, and as per CPCB norms, recycling of backwash wastewater has been adopted by many plants/utilities. Reuse of filter backwash water is recommended for water treatment plants above 5 MLD capacity for economy and operation and maintenance.

Filter backwash water should be recycled to inlet chamber or upstream of flow measuring flume or weir, and to be done as uniformly as possible. It should not increase more than 2% to 6% of plant flow rate. It requires a wash water re-circulation tank of adequate volume (equivalent to the storage capacity of elevated wash water tank) to receive and store the filter backwash water. For plants less than 30 MLD capacity, one tank is sufficient. For larger plants, one tank with two compartments shall be provided. If the air grid or agitator is provided for the tank, then it eliminates the need for manual de-silting.

The recycle rate needs to be worked out carefully, taking into consideration the required redundancy. Normally, recycle pumping hours are 16 to 20. To reduce the re-circulation shock loading (to mainstream), it is better to have more number of pumps of smaller capacity. Vertical or horizontal centrifugal pumps can be provided for recycle. Normally the location of wash water re-circulation tank is at the lowest available contour at site. Because of the plant hydraulics, it is experienced that abnormally high freeboards are required to be provided to receive the wastewater from filters by gravity.

There is no need for clarification or settling of recycle back wash water if the quantity is limited to 2% to 6% of total flow. Excessive recycle rates may lead to change the characteristics of raw water.

In some countries, there is a practice of disinfecting the recycle water by chlorine or ozone to eliminate Giardia lamblia cysts, Cryptosporidium cysts and to inactivate viruses. However, this may lead to formation of DBPs and this practice should be avoided.

The sludge from clariflocculator is not mixed with backwash water from filters. Therefore, the entire backwash water should be recycled. This will not only minimise the losses through treatment plant, but also enhance the removal efficiency. Though, theoretically outlet to this backwash recycle tank is not needed, however, in emergency (if both the pumps, working as well as standby fail to operate), the outlet for this tank is needed to prevent them to overflow/flood the pump floor.

In developed countries, the backwash recycle water is treated trough the tube/plate settlers and chlorinated to eliminate suspended solids, Giardia, Crypto schists and schists before it is sent to the inlet chamber of the plant.

Figure 8.49: Wash Water Recycle Tanks (Single Compartment and Two Compartments)

8.9 Disposal of Wastes and Sludge from Water Treatment Processes

Disposal of wastes from the water treatment plants has become increasingly important with the availability of technology and the need for protection of the environment. The present CPCB norms do not allow more than 100 mg/L of total suspended solids (TSS) to be disposed into water bodies. Treatment of waste solids adds to the cost of construction and operation of treatment plants. Sludge from water treatment plants comprise of:

(a) Sludge from sedimentation of particulate matter in raw water, flocculated and precipitated material resulting from coagulation, or residuals of excess coagulant dosage, plankton, etc.
(b) Sludge generated from reactions of coagulants. However, these quantities are minor.
(c) Sludge generated from lime-soda softening processes and silica removal.
(d) Wastes from regeneration processes of ion exchange softening treatment plant containing cations of calcium, magnesium and unused sodium and anions of chlorides and sulphates originally present in the regenerate.
(e) The primary focus of the discussion in this chapter is on sludge generated by plants with surface water sources (a). The plants based on lime-soda softening plants (c) and ion-exchangers (d) are not very common in public water supply systems or they are limited to very small plants in India.

The reuse of alum/PAC sludge is possible, however, techno-economic feasibility of recovery of alum/PAC/Alumina is only feasible for exceptionally large plants.

8.9.1 Disposal Methods

As a result of the presence of suspended and precipitated solids, as well as reactions with many coagulants, the process generates sludge. The quantity of sludge generated is directly proportionate to total suspended solids (TSS) in raw water. Most of the sludge (90% to 98%) is effectively removed from water in different clarification units such as simple sedimentation tanks, conventional clarifiers, clariflocculators, tube settlers, lamella clarifiers, and the like. The separated sludge contains almost 92%–95% water and just 5%–8% insoluble/inert solids and precipitates.

Clarifiers or settling tanks shall be a single point source to take out the sludge from the system for treatment and disposal. The dirty backwash water shall have a separate stream for recycle as described above sections. It shall not be mixed with clarifier sludge for further thickening.

Various types of clarifiers and settling tanks produce sludge of different consistency (also called as an underflow) depending on TSS in raw water. The sludge consistency from clarifiers/settling tank varies from 2% to 6% W/V. For designing the thickening system, it is recommended to adopt value of 2%. The principle of sludge treatment is progressive thickening of the sludge from dilute turbid liquid to moist solids cake with consistency 20% to 25% W/V.

Equation for sludge/solids mass balance for clarifier (as well as for thickener) is as follows.

Total suspended solids (TSS) rate at Inlet to system (mg/hr.) =
Total suspended solids (TSS) rate at outlet of the systems (mg/hr.) + Total suspended solids (TSS) rate in sludge flow (mg/hr.)

Simplified as follows to find sludge flow rate in cum/hr.

[Inlet flow (m^3/hr.) × Inlet TSS (mg/L)] =
[Inlet flow (m^3/hr.) – Sludge flow (m^3/hr.)] × Outlet TSS (mg/L)] + [Sludge flow (m^3/hr.) × Sludge TSS (mg/L)

8.9.1.1 Gravity Sludge Thickener

This sludge from clarifiers or settling tanks shall be further thickened by using gravity thickener. In some cases, it can be directly led to centrifuges or sand drying beds. For thickener, the design criteria are 60–80 kg/m^2/d (max up to 100 kg/m^2/d) based on the solids loading (dry basis) and is followed by sludge dewatering devices. A thickener is similar to a central feed, central rake drive clarifier having depth of 3.5 m to 4.0 m. The rake tip speed is 1.0 m/min to 2.0 m/min. For inlet sludge (underflow) consistency of 2% (W/V), the thickener underflow consistency is in the range of 3% to 6% W/V. For design purpose 5% W/V value is recommended. The lime sludge has shown better dewatering characteristics. PE dosing is practised at the inlet of the thickener to improve the quality of the underflow. The thickener rake has normally pickets mounted on it (Vertical angles or supported flats). Theses pickets help in releasing the water interlocked between the solid's particles in the hindered settling (bottom sludge) zone. The scraping mechanism has to be coupled with high-torque speed reduction mechanism.

The underflow (thickened sludge) coming out of thickener shall be further de-watered by processing it through sand drying beds, centrifuges, or presses. The consistency of dry cake coming out of filter presses can be up to 20% to 25%, while that from centrifuges is up to 15%–20%. Freezing and thawing method improves the sludge consistency. However, it is not very relevant to our public water sector systems. The supernatant from the sludge thickener is recycled to the backwash water recycle tank/sump.

8.9.1.2 Sludge dewatering devices

A variety of dewatering devices/systems are employed, including but not limited to:

1. sludge drying beds (SDBs);
2. continuous decanter centrifuges;
3. batch type filter presses;
4. continuous filter presses;
5. batch type lagoons.

The IS 10037-2 (1983, reaffirmed 1996): Requirements for sludge dewatering equipment, Part 2: Vacuum filtration equipment, which lays down requirements for vacuum filtration equipment used for sludge dewatering may be referred to.

8.9.1.3 Sludge Drying Beds (Sand Beds)

The sand (sludge) drying beds for dewatering occupy a huge area. Their performance is also affected by climatic conditions. They have their use in dry, arid climate, where rainfall is less or moderate. The area requirement for these beds is based on solids loading criteria of 150 to 400 $kg/m^2/d$. The drying cycle is designed for six to seven days. However, both these criteria are dependent on the weather and climate of the region. The depth of sludge application is restricted to 200 to 400 mm. The sand depth is 300 mm to 400 mm with E.S. varying from 0.3 mm to 0.75 mm and U.C. in between 3 to 4. The sand is supported over gravel/pebble of depth 200 mm to 250 mm. Perforated laterals of cement pipes are preferred to convey the filtrate up to central manifold. The bottom slope of the beds is 1:100 towards manifold pit. With bright sunshine for at least 8–10 hours a day, and with proper sand gradation, it is possible to achieve solids concentration of 40%–60% in the dried cake. Sludge drying bed are recommended for plants less than 5 MLD capacity.

8.9.1.4 Sludge Drying Beds (Tile Beds)

Tile bed systems have been designed to effectively dewater both industrial and municipal sludges. Basins and bed sizes can be adjusted to fit any application, including upgrading an existing sand bed or converting lagoons. The system is flexible enough to handle industrial spikes or heavy surges from digesters and holding tanks. Tile systems are designed to meet the ever-changing demands of the water and wastewater industries by providing an environmentally responsible dewatering system. The operation is simple: flood the bed with thickened sludge, wait for the sludge to dry, and remove the dry sludge cake with a front-end loader. Following sludge removal, wash down the bed and repeat the cycle.

8.9.1.5 Continuous Decanter Centrifuges

A decanter centrifuge operates on the principle of solid-liquid separation based on enhanced difference in specific gravity when a centrifugal force is applied on the mixture. A decanter centrifuge separates solids from liquid phase in a continuous process. The denser solid particles are pressed outwards against the rotating bowl wall, while the less dense liquid phase forms a concentric inner layer.

Different ‘dam plates’ are used to vary the depth of the liquid – the so-called pond – as required. The centrifugal force compacts the solids and expels the surplus liquid. The dried solids then discharge from the bowl. The clarified liquid phase or phases overflow the dam plates situated at the opposite end of the bowl (Figure 8.50 & 8.51).

Figure 8.50: Centrifuge Building with "Chute"

Figure 8.51: Centrifuges located on the First Floor

8.9.1.6 Batch Type Filter Presses

Filter presses are also used on a large scale for dewatering of sludge. These machines are also referred to as 'plate and frame' filter presses. A filter press is a batch operation, fixed volume machine that separates liquids and solids using pressure filtration. A slurry is pumped into the filter press and dewatered under pressure. It is used for water and wastewater treatment in a variety of different applications ranging from industrial to municipal. Hence this press operates in 'cycles'. Cycle time typically consists of filling, filtration against pump pressure, further draining though air pressure, plates opening, sludge removal (may be manual or auto) and re-assembly. At times, even water wash is also provided for further extraction of soluble substances, if required by the application. Typical cycle time can vary from about two hours and may extend up to even eight hours or beyond, depending on the drainability of the sludge, applied pressure, type of filter cloth selected, etc.

Though most of the dewatering takes place under feed pressure, for further drying of sludge (cake), many times, pressurised air is passed through the plates, to forcefully expel the moisture. In such cases, a higher dryness cake can be obtained. Designer must remember that this is a batch operation, hence once the filter press is loaded, there must be enough storage volume available for sludge storage, or a second working press is to be provided.

8.9.1.7 Continuous Filter Press

A continuous filter press is a unit in which sludge is squeezed between two continuous travelling porous belts (sometimes referred to as 'felts'). Multiple rollers are provided which progressively press the felts, thereby draining moisture from the sludge. Dewatered sludge is then discharged from the final roller or scraper. Usually, to keep the felts clean, and to remove any sludge particles stuck to felts, pressure water is sprayed from both sides of both the felts, so that porosity of filter media is maintained.

The advantage of such belt filter press over the plate and frame press is the continuous operation. However, such continuous filter presses demand higher maintenance, higher freshwater consumption (for felt washing). Continuous filter presses can be more compact as compared to batch type filter presses, and usually there is no standby required, except few essential spares.

8.10 Treatment Plant Hydraulics

To the extent possible, the flow through the main process units shall be based on gravity. Generally, it is observed the hydraulic losses in the treatment plant vary from 4.50 m to 6.50 m, the bigger the plant, the more the head loss. In the steeply sloping or hilly terrain, the head loss is likely to increase due to lack of space to suit the designed hydraulic gradient. The reason for restricting the total losses through the plant is conservation and saving of overall energy.

As far as safe practices are concerned, provision of the following minimum losses are recommended.

Rapid Mix Unit (Flash Mixer) to Clarifier = 0.40 m
Clarifier to Filter Bed = 0.40 m
Filter Pure Water Channel to Pure Water Sump = 0.40 m.

The treatment plant hydraulics comprises of open channel and piping (closed conduits) losses, valves, gates, pipe specials, free-falls and terminal head loss of filter beds. For open channels and drainage systems, it is recommended to use the Manning formula. For piping and closed conduits, it is recommended to use the Hazen-Williams formula. Valves, gates, and pipe specials are governed by $h = K (V^2 / 2 \times g)$, where h = losses in m, V is the velocity in m/s, g is the gravitational constant, and K is the coefficient or the multiplying factor. The value of K may vary slightly in different reference books; however, it is important that when one value is selected, it shall be uniformly applied to the entire system.

As a good engineering practice, it is recommended to design open channels, piping, and closed conduits for 20% overloading factor over the rated design capacity. (Please note that process units are not to be sized for overloading) It is recommended that open channels shall be designed for low velocities to avoid breaking or fragmenting the floc particles. For piping the velocities are recommended in the range of 0.8 m/sec to 1.2 m/sec. Generally, the Full Supply Level (FSL) of pure water sump or clear water Reservoir/sump (CWR) shall not be below the finished ground level (FGL). However, it may not be possible in a few exceptional cases. The PW sump and CWR shall be provided with an overflow arrangement, either a bell mouth pipe or sharp crested weir.

Whenever there are two units provided in parallel, the flow shall be divided equally by providing split flow weir arrangement on the upstream in order to ensure equal distribution. This, i.e., especially true for the rapid mix unit (flash mixers) and clarifiers. Free-falls in the plant shall be provided judiciously to avoid the flooding of upstream units. The maximum values of filter port velocities are shown in Table 8.10.

Table 8.10: Maximum values of Filter Port Velocities

Filter Port	Velocity (m/s)	Adopted in Maharashtra
Filter Inlet	0.9 to 1.8	1.2
Filter Outlet	0.9 to 1.8	1.2
Wash Water Inlet	2.4 to 3.65	3.0
Wash Water Outlet	2 to 2.4	2.5
Air Scour Isolation	20 to 25	25

Source: Water Supply & Waste Water Disposal by Fair & Geyer

8.11 Layout of Water Treatment Plants

Normally, water treatment plant complex, apart from the process units, include ancillary structures like filter annex building, chemical (coagulant, etc.) house and store, MCC/switch room, chlorination room, tonner yard, generator room, transformer yard, office, staff quarters and security cabin, etc. Many at times it includes elevated service reservoir or MBR also. The entire plant complex should be planned in such a way that process and hydraulic structures are located in a sequential and logical way to reduce piping and interconnecting channels. Due consideration should be given to the convenience of operator movement. All the structures should be interconnected by walkway at the operating level. The roads and pathways should be provided on the ground, interconnecting all the units. It is advisable to club together main treatment unit structures on one side and waste recycle and treatment structures on the other side. All ancillary structures should be located to complement the process and hydraulic structures with respect to their functions. The overall layout should be aesthetically pleasing by providing garden in the vacant spaces and with trees on the plot boundary.

Ideally, a site with mild slope is most suitable to ensure minimum excavation. The ideal soil bearing capacity is 10 to 15 MT/m^2. As far as possible, sites with exposed hard rock, black cotton soil, and water-logged sites, should be avoided. The plant should be located near the community and not in the isolated places for better supervision and control. However, in practice, such ideal conditions are rarely available, especially in the hilly terrain. In many cases adequate area is not available due to cost constraints. There also could be obstructions at site like existing structures, buried pipelines and conduits, overhead HT cables, road, rivulets, etc. Hence, it becomes a challenging job for the designer engineer. The design engineer needs to study the contours (1 m interval) of the land and hydraulic flow diagram to locate various units in an economical manner to prepare the layout plan. A very general rule is to place the structures with higher elevations on higher contours and structures with lower elevations on lower contours so that their bottom slabs are on a sound foundation. Wherever there is a double pumping scheme (raw as well as pure water), the engineer has some freedom to select the levels of the structures. But the constraints start building up when gravity flows (to and from the plant) are necessitated by the overall scheme hydraulics. The site selection becomes extremely critical if it is a double gravity scheme (raw as well as pure water).

The layout of the plant complex should take into account the future expansion of the plant. Micro-planning involves routing the buried pipelines, plant drainage, site drainage, routing of piping, cable trays, planning interconnections, headroom clearances, operator safety, etc. Adequate architectural features should be incorporated in the plant buildings to impart the prominence and importance of the plant to the community.

Typical Layouts of WTPs for various capacities

Figure 8.52: Layout of a small capacity plant of 10 MLD capacity on a fairly levelled site (double pumping scheme)

Figure 8.53: Layout of a medium capacity plant of 25 MLD on a steeply sloping hill (raw water by pumping, PW supply by gravity)

Figure 8.54: Layout of a medium capacity plant of 50 MLD capacity with hard rock as strata (double pumping scheme)

Figure 8.55: Layout of a large capacity plant of 168 MLD with severe space constraints

Approximate land area requirement (Table 8.11) for conventional water treatment plants consisting of cascade aerator, Parshall flume, flash mixers, clariflocculators, rapid sand gravity filter beds, filter annex building, storeroom, wash water recycling tank, sludge thickener, centrifuges, MCC room, generator room, transformer yard, etc.

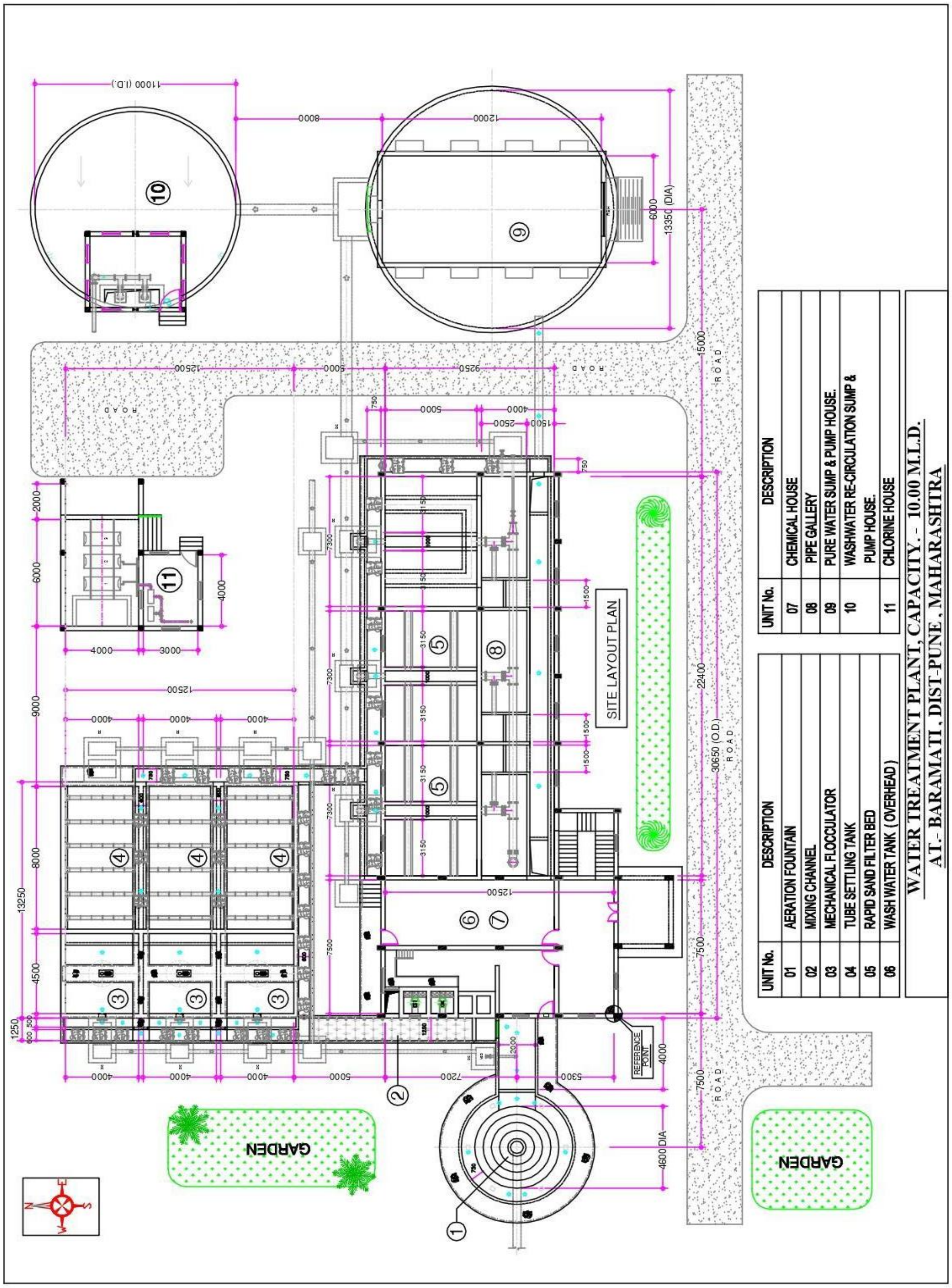


Figure 8.52: Layout of WTP Capacity 10 MLD at Baramati, Dist. Pune, Maharashtra

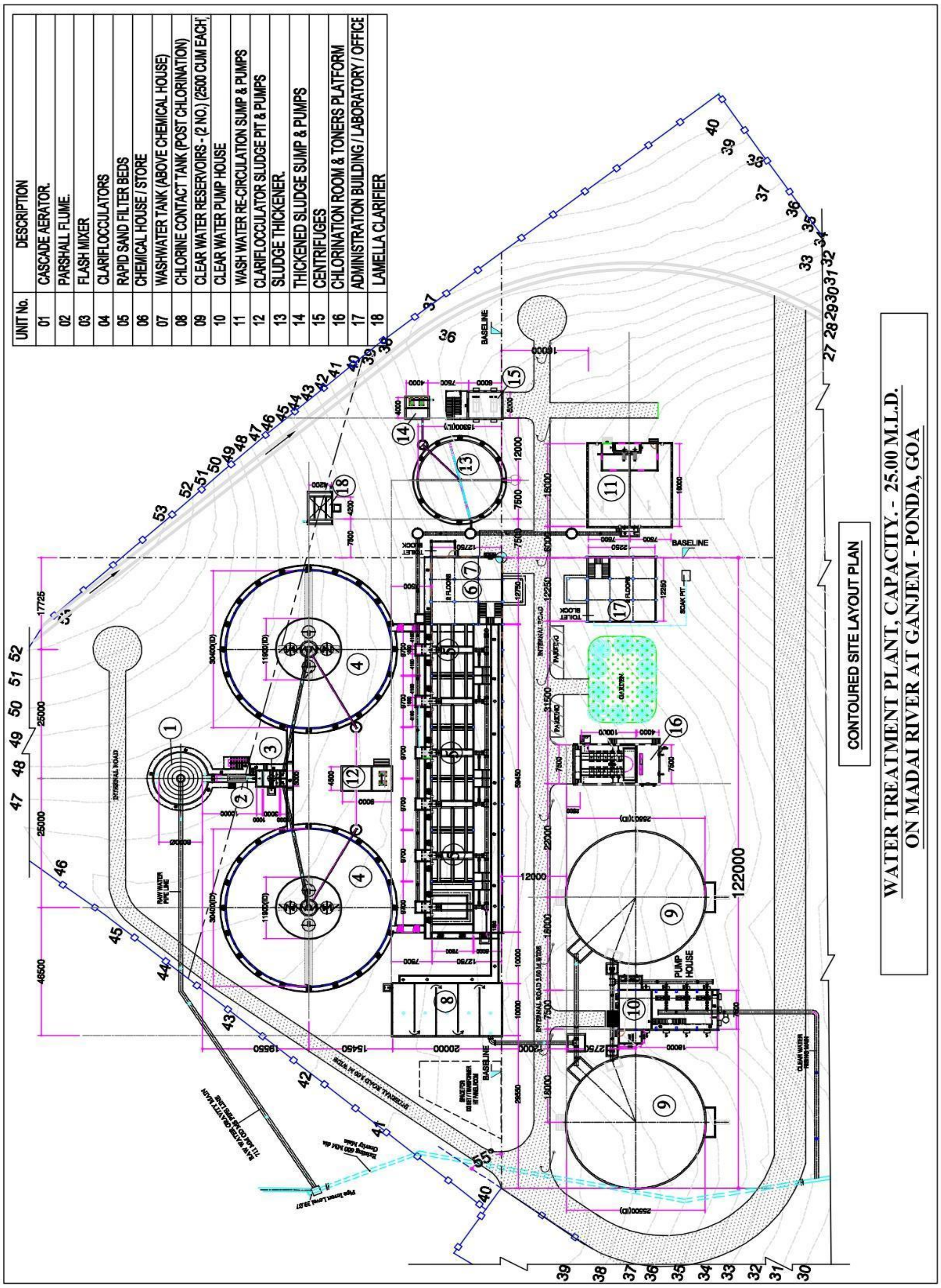


Figure 8.53: Layout of WTP Capacity 25 MLD at Ganjem, Ponda, Goa

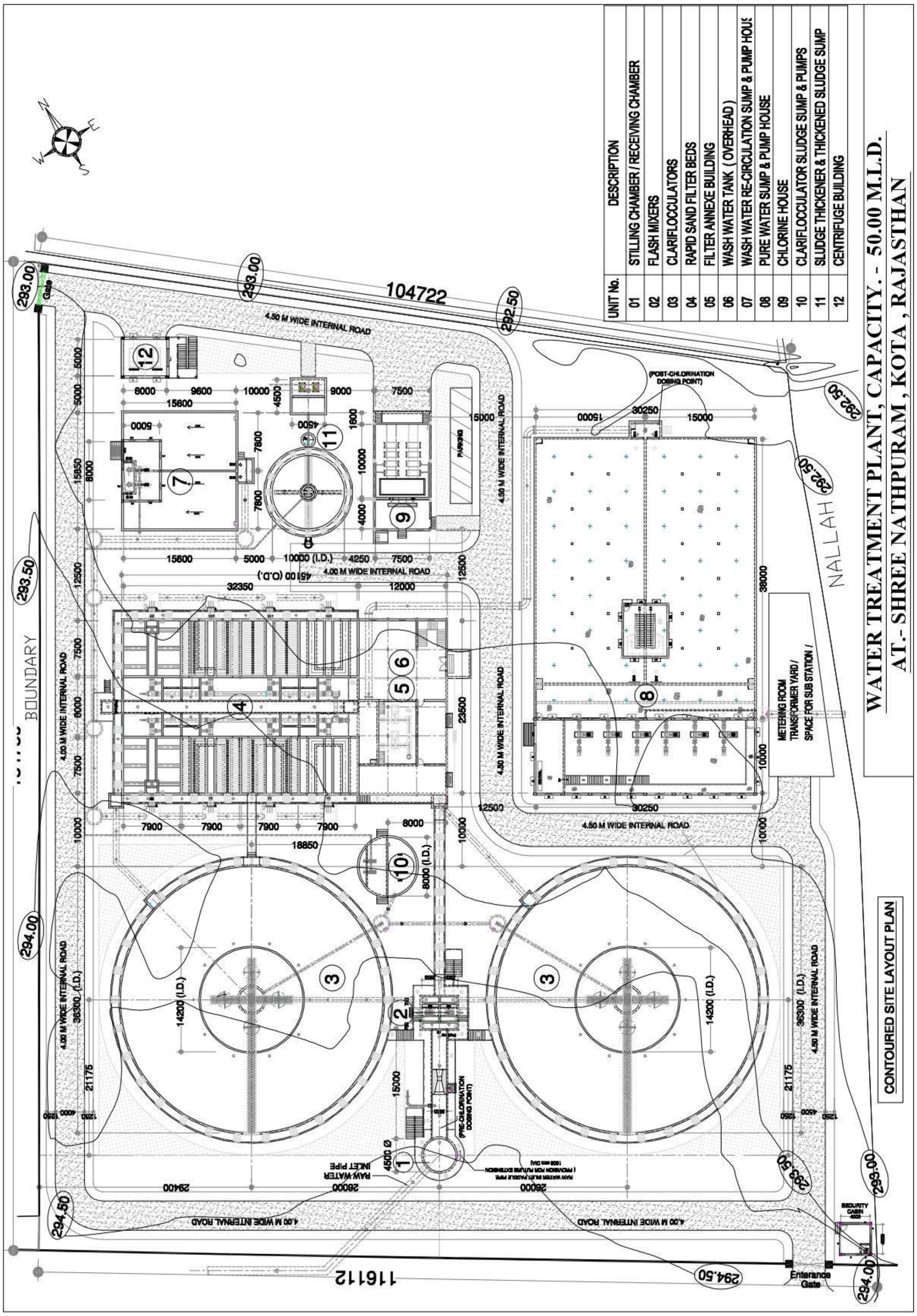


Figure 8.54: Layout of WTP Capacity 50 MLD at Shreenathpuram, Kota, Rajasthan

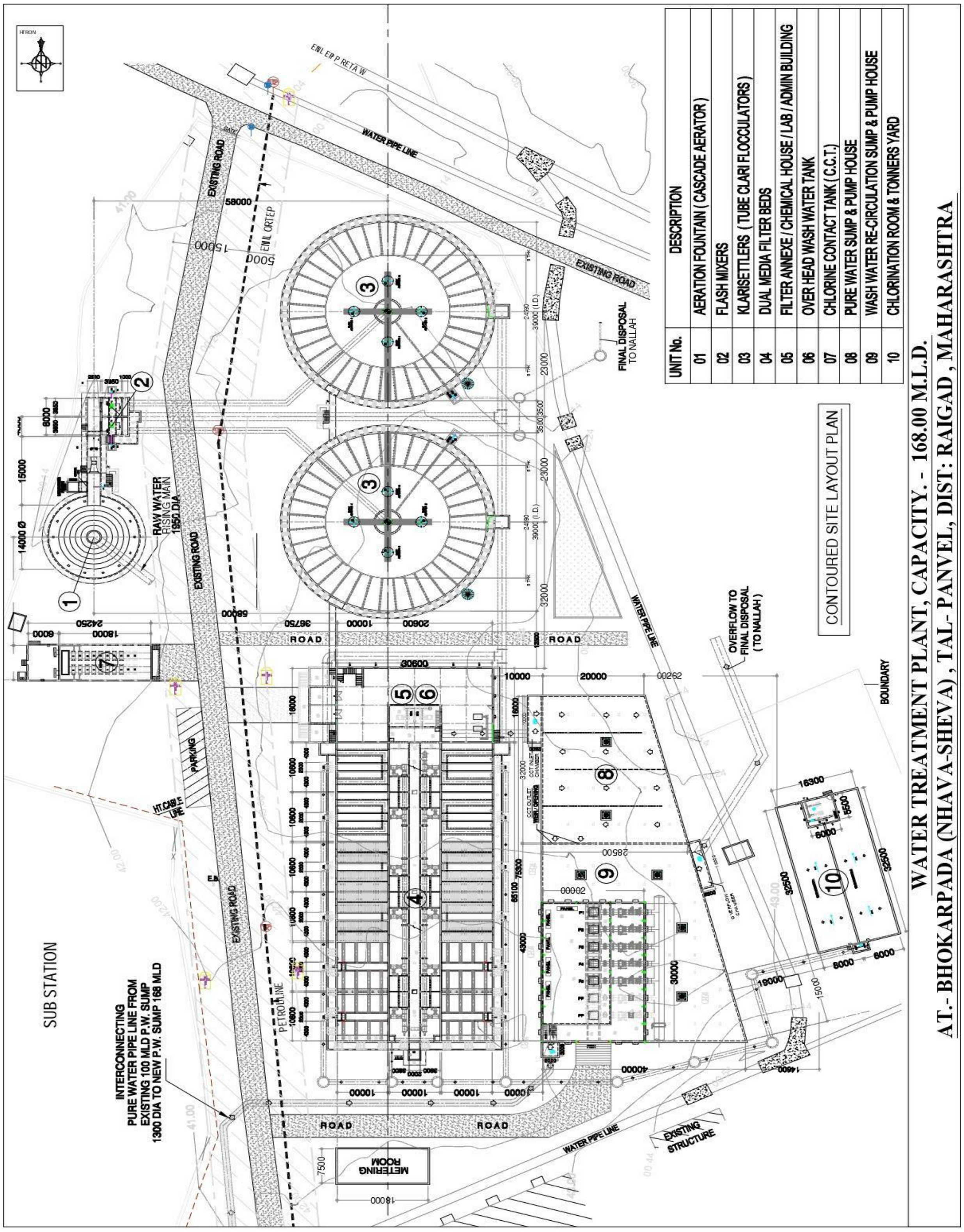


Figure 8.55: Layout of WTP Capacity 168 MLD at Nhava-Sheva, Dist. Raigad, Maharashtra

Table 8.11: Approximate Land Area Requirement for Conventional Water Treatment Plants

S. No.	Plant Capacity (24 hr. basis) (MLD)	Area in sqm	Area in acre (average)
1.	1.0	1,000–1,200	0.30

S. No.	Plant Capacity (24 hr. basis) (MLD)	Area in sqm	Area in acre (average)
2.	5.0	2,500–3,000	0.70
3.	10.0	4,500–5,000	1.15
3.	20.0	8,000 –10,000	1.75
4.	50.0	20,000–22,000	3.00
5.	100.0	25,000–27000	6.50
6.	200.0	60,000–70,000	16.00

8.12 Augmentation or upgradation of Existing Water Treatment Plants

Augmentation of the existing plant can be considered if the life of the WTP has not approached to its design life (30 years), but there is a disproportionate increase in demand and there is a constraint of space to construct the new structures.

Augmentation can be achieved either by addition of new plant or by retrofitting all the units in the existing plant with the appropriate technologies, such as installation of plate/tube settlers, high-rate gravity filters, disc filters, and low-pressure submersible membranes.

Upgradation of existing treatment plant can be considered and redesigned by retrofitting the existing units if there is change in raw water characteristics and the existing plant is not able to produce treated water of desired quality, such as in the case of an increase in nitrate due to discharge of wastewater upstream.

The existing water treatment plants can be augmented (capacity-wise) or upgraded (quality-wise) by retrofitting the conventional process units with high rate and advanced unit processes (described earlier). Simultaneously, hydraulics of the plants are also required to be managed for the enhanced capacity. The cost of the augmented plant (say, to double the capacity) is 60% to 70% that of a new plant. In addition, there are savings in the precious land cost too. The augmented plant can be operated with the existing manpower. There is only marginal increase in electrical power. During retrofitting the machinery of the existing plants gets overhauled, increasing the longevity of the plant. Generally, it has been experienced that the existing conventional plants consisting of flash mixers, clariflocculators and rapid sand gravity filters can be augmented to twice their original capacity or more.

Conceptual Design considerations:

(a) **Flash mixers and flocculators:** Velocity gradient (G) and detention time (t) are the governing factors for the design. By enhancing the flow through the unit, the detention time reduces. To supplement this, velocity gradient needs to be increased to main the ‘Gt’ value as per the norms.

(b) **Clarifiers or Clariflocculators:** The surface loading rates of tube settlers four to five times that of conventional clarifier. Hence these units can be retrofitted with tube settlers to enhance the capacity. Usually, part of the outer annular zone of clarifier is partly covered with the module (Figure 8.56 & 8.57).

(c) **Rapid Sand Gravity Filter Beds:** The rate of filtration of dual media gravity filters and mono media deep bed filters is two to three times that of rapid sand filters. These filter beds can be retrofitted with high-rate filter media configuration (Figure 8.58).

(d) **Hydraulics:** The interconnecting piping, channels, inlet and outlet ports, if required, need to be redesigned to maintain the head loss between any two units same as per the existing plant. Inherent redundancy in the sizing of the existing piping and channels need to be utilised judiciously.

Figure 8.56: Tube Modules installed in Clariflocculator

Figure 8.57: Retrofitting of a rectangular tank with tube settle

Figure 8.58: Upgrading of Existing conventional filter beds with mono media deep bed filters

8.13 Prefabricated Packaged Water Treatment Plants

These are installed mostly in small urban town and village water supply schemes which have less than 5 MLD capacity (Figure 8.59). These are constructed in an environment where skilled manpower is not available. These schemes are mostly executed by the local contractors, and they require guidance on all aspects of construction. In-situ concrete or civil plants take long time to construct, and execution is error-prone. For such small plants, prefabricated packaged plants are recommended, where the quality of construction can be assured in the controlled environment. The emphasis is on providing time-tested unit processes with the least mechanical or rotary parts. These shall be fabricated in mild steel plates of 6 mm thickness. The inside of the plants (surface coming in contact with water) shall be provided with protective lining of FRP with 5 mm thickness. From outside, the surface shall be painted with primer and two coats of zinc-based paint. These plants shall be designed to be operated manually. The size of the units shall be such that these can be transported by standard trucks easily. Only pure water sump, pump house, and control room shall be constructed in RCC in-situ construction. Based on experience, life of such plants can be considered to be 15–20 years.

Following unit processes and design criteria are recommended.

(a) Rapid mix weir or 90° V-notch with flow measurement, mixing channel.
(b) Pebble (gravel) bed, plastic tetrapod, or conventional paddle type flocculator with detention period of 15–20 min.
(c) Tube/Plate settling tank with hopper, with surface loading rate less than 6.5 $m^3/m^2/hr$.

(d) Rapid sand gravity filters with rate of filtration 5.5 $m^3/m^2/hr$.
(e) Disinfection by calcium or sodium hypochlorite (bleaching powder).

Figure 8.59: Prefabricated Packaged Water Treatment Plants

8.14 Computer-Aided Optimal Design of Water Treatment System

(i) The advent of computerised design systems/methodology has drastically changed the design process of water supply system including water treatment plant design. Many companies have developed in-house design programmes for optimising every step of water treatment plant design.

The water treatment process involves sizing, hydraulics, electro-mechanical aspects, as well as instrumentation, and they are interlinked. The optimal design aided by iterative process of computer programming helps in adhering to all the design/code specifications and come out with a best possible size of various components and machinery.

It is recommended to use computer-aided optimal design to construct the plant which ultimately will be useful in effective operation and maintenance including automation and SCADA.

Computer-aided design of water treatment plants only generate sizing and losses calculations. However, in practice, the designer must submit the site layout, hydraulic flow diagram and general arrangement drawing of all the units showing all the detailing not generated by the design programme. Though the computer-aided design can reduce the time to some extent, it cannot substitute the experience of the designer.

(ii) Prequalification for Designer of Water Treatment Plants (Hydraulics and Process)

The designer must hold a master’s degree with 10 years of experience, or a bachelor’s degree with 20 years of experience in civil, environmental or chemical engineering. The designer shall have minimum experience of designing one plant of the same (rated) capacity, or two plants of half capacity, or three plants of one-third capacity. The designer shall provide a certificate from competent authority of satisfactory completion and approval of documents of his past jobs. Information to be included in tender specification is enclosed in **Annexure 8.3.** Summary of recommended design criteria for WTP is enclosed in **Annexure 8.5**.

The same prequalification criteria are suggested for the approving authority.

(iii) Pilot Plant Studies

When selecting high-rate technologies or units, to achieve the optimum performance it is prudent to run pilot plant trials. Many equipment manufacturers have scaled down models which they can transport to the site. Pilot plant testing can optimise the design values of various parameters to suit the specific raw water source. This is especially true for high-rate filtration where media configuration, media depth and grain size can be optimised to extract the best performance.

CHAPTER 9: DISINFECTION

9.1 Disinfection

For the utmost safety of water for drinking purposes, disinfection of water has to be done in order to kill disease-causing microorganisms. Bacteria, viruses, and protozoa constitute the three main types of human enteric pathogens. Accordingly, effective disinfection with adequate detention time is aimed at the destruction or inactivation of pathogens. The need for disinfection in ensuring protection against transmission of water-borne diseases cannot be overemphasised and its inclusion as one of the water treatment processes is considered necessary.

Historically, boiling of water or the use of copper and silver vessels for storing water which affects some measure of disinfection has been employed at small scale in this country and elsewhere. Broadly, modern disinfection processes include the use of:

(i) Physical methods such as thermal treatment and ultrasonic waves.
(ii) Chemicals including oxidising agents such as chlorine and its compounds, bromine, iodine, potassium permanganate, ozone, and metals like silver.
(iii) UV radiation.

9.1.1 Mechanisms of Disinfection

The mechanism of killing the pathogens depends largely on the nature of the disinfectant and the type of microorganisms. In general, four mechanisms are proposed to explain the destruction or inactivation of organisms.

(i) Damage to the cell wall.
(ii) Alteration of cell permeability.
(iii) Changing the colloidal nature of the cell protoplasm.
(iv) Inactivation of critical enzyme systems responsible for metabolic activities.

Damage to the cell wall leads to cell lysis (disintegration of cell by rupture of cell wall or membrane) and death. Alteration of cell permeability refers to the destruction of selective permeability of the cytoplasmic membrane and causes outflow from the cells of such vital nutrients, like nitrogen and phosphorus. Denaturation of cell proteins by acids and bases leads to the destruction of cells. Inactivation of critical enzyme activity vital for cell growth and survival is normally brought about by oxidising chemicals. Figure 9.1 shows the mechanism of disinfection.

Chemical disinfection normally proceeds in at least two steps:

(i) Penetration of the disinfectant through the cell wall; and
(ii) Reaction with enzymes within the cell.

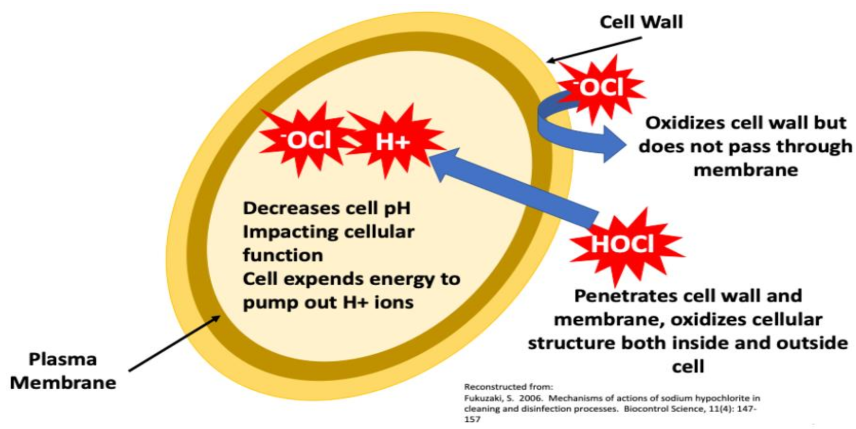


Figure 9.1: Disinfection Mechanism (source: Fukuzaki, S. 2006)

9.2 Criteria for a Good Disinfectant

For a chemical or an agent to be potentially useful as a disinfectant in water supplies, it has to satisfy the following criteria:

(i) Be capable of destroying the pathogenic organisms present, within the contact time available and not unduly influenced by the range of physical and chemical properties of water encountered particularly temperature, pH, and mineral constituents.

(ii) Should not leave by-products of reaction which render the water toxic or impart colour otherwise will make it unpotable.

(iii) Possess the property of leaving measurable residual concentrations to deal with possible recontamination.

(iv) Be amenable to detection by practical, rapid, and simple analytical techniques in the small concentration ranges to permit the control of the disinfection process.

The efficiency of chemical disinfection is influenced by the following factors:

(i) Type, condition, concentration, and distribution of organisms to be destroyed.

(ii) Type and concentration of disinfectant.

(iii) Chemical and physical characteristics of water to be treated.

(iv) Contact time available for disinfection.

(v) Temperature.

9.3 Type, Condition, and Concentration of Microorganisms to be Destroyed

A chemical disinfectant has to diffuse through the cell wall before it can react with the enzyme systems. Since the different types of organisms have different cell structures and enzyme systems, the action of the disinfectant varies accordingly. Among intestinal organisms, pathogens are less resistant than the coliform group and hence, the latter can serve as a convenient index of the efficiency of disinfection.

Viruses appear to be more resistant than bacteria and require longer periods of contact as well as a higher concentration of disinfectant. The condition in which the organisms occur may also affect the efficiency of disinfection. Thus, when the bacteria are clumped together, the cell inside the clump may be protected against the action of disinfectant. The density of the organisms affects efficiency when the number is so high that there is a deficiency of available disinfectant. Such a condition may occur in the disinfection of sewage but is not usual in water works practice.

9.3.1 Type and Concentration of Disinfectant

The efficiency of disinfection will obviously depend on the nature of the disinfectant. The added chemical undergoes several transformations so that the disinfecting action is exerted by the end products of the reaction. The course of these reactions is largely influenced by the character of the water and its constituents. These reactions that may occur under different conditions will determine the type and proportion of the active disinfectants. The higher the concentration of a chemical disinfectant, the higher the destruction of organisms.

9.3.2 Chemical and Physical Characteristics of Water to be Treated

Chemical and physical characteristics of water affect the reach of the disinfectant and thus its efficacy. For example, organic matter and certain oxidising constituents in water reduce the availability of active products for disinfection. Embedded organisms in suspended materials in water may be sheltered from the action of disinfectant.

9.3.3 Time of Contact available for Disinfection

The destruction of organisms increases with the contact time available for disinfection. In practice, the contact period is limited by the design of the water treatment plant.

9.3.4 Temperature of the water

Rates of chemical reactions are speeded up as the temperature of the reaction is increased. The higher the temperature, the more rapid is the destruction of organisms.

9.4 Mathematical Relationships Governing Disinfection Variables

The kinetics of disinfection are affected by several variables. The effect of some of these disinfection variables can be quantified by empirical mathematical relationships. Under ideal conditions, three main variables alter the rates of disinfection, namely (i) the time of contact, (ii) the concentration of the disinfectant, and (iii) the temperature of water.

9.4.1 Contact Time

Contact time is an important variable affecting the rate of destruction of organisms, generally speaking, under ideal conditions and at a constant temperature. The number of organisms (N_t) surviving after a period of time t is related to the initial number (N_o) by

Chick's law,

$$log\frac{No}{Nt} = k.t \qquad (9.1)$$

Where k is constant with dimension (T^{-1})

Departures from Chick's law are not uncommon; rates of kill have been experimentally observed to increase with time in some cases and decrease with time in other cases. To account for these departures from Chick's law, the following modified equation has been suggested:

$$log\frac{No}{Nt} = k.t^{m} \qquad (9.2)$$

Where m is a constant. If m is less than 1, the rate of kill decreases with time and if m is greater than 1, the rate of kill increases with time. Laboratory analysis and subsequent interpretation of data may provide useful information for design purposes.

9.4.2 Concentration of Disinfectant

The rate of disinfection is affected, within limits, by changes in the concentration of disinfectant. The relationship between disinfectant concentration and the time required for killing a desired percentage of organisms is generally expressed by the following equation:

$$C^n \times t_p = \text{Constant} \tag{9.3}$$

Where C is the concentration of disinfectant, n is a coefficient of dilution and t is the time required for a constant percentage kill of the organisms.

9.4.3 Temperature of Water

At a lower temperature of water (e.g., in the winter season) the time required for achieving the same percentage of kill for the same concentration of disinfectant would be higher than those for a higher temperature (e.g., in summer months). If the time of contact cannot be changed due to design constraints, doses of disinfectants will have to be changed to account for changes in temperature to achieve the same percentage of kills.

9.5 Chlorination

Chlorination is the most widely used method for disinfecting water supplies across the world. The near universal adoption of this method can be attributed to its convenience and to its highly satisfactory performance as a disinfectant, which has been established by decades of use.

9.5.1 Chlorine Demand

Chlorine and chlorine compounds, by virtue of their oxidising power, can be consumed by a variety of inorganic and organic materials present in the water before any disinfection is achieved. It is, therefore, essential to provide sufficient time and dose of chlorine to satisfy the various chemical reactions, and leave some amount of unreacted chlorine as residual, either in the form of free or combined chlorine, adequate for killing the pathogenic organisms.

The difference between the amount of chlorine added to water and the amount of residual chlorine after a specified contact period is defined as the chlorine demand. The chlorine demand of any given water varies with the amount of chlorine applied, the time of contact, pH, temperature, and form of residual desired.

9.5.2 Chlorination Practices

The type of available chlorine residual required, and the characteristics of the water being treated, determine the process of disinfection to be employed. All chlorination practices, irrespective of the point of application, may be classified as free available residual chlorination (i.e., break point or super-chlorination) or combined residual chlorination, depending on the nature of the chlorine residual formed.

9.6 Free available residual Chlorination

9.6.1 Plain or simple chlorination

This involves the application of chlorine to water as the only type of treatment to offer the necessary public health protection. Plain chlorination can be carried out in a situation where:

(i) Turbidity and colour of the raw water are low, turbidity should not be exceeding 5 NTU;

(ii) Raw water is drawn from relatively unpolluted sources;

(iii) Water contains little organic matter, and iron and manganese do not exceed 0.3 mg/L; and

(iv) A contact period of a minimum of 30 minutes between the point of chlorination and the consumer end is available.

9.6.2 Super-Chlorination

This is adopted in case of an emergency situation such as a breakdown or in case of waters that are heavily polluted or fluctuate rapidly in quality. It can give excellent results in waters where:

(i) plain chlorination produces taste and odour;

(ii) the water is coloured; or

(iii) iron and manganese have to be oxidised.

It may be resorted to on special occasions when available contact time is limited at the pre-chlorination stage. Super-chlorination can effectively destroy relatively resistant organisms such as viruses and amoebic cysts. The dose of chlorine may be as high as 10 to 15 mg/L with contact periods of 10 to 30 minutes. Subsequently, the undesirable excess chlorine will have to be dechlorinated.

9.6.3 Breakpoint Chlorination

The addition of chlorine to ammonia in water produces chloramines which do not have the same efficiency as free chlorine. If the chlorine dose in this water is increased, a reduction in the residual chlorine occurs, due to the destruction of chlorine by the added chlorine. A few possible chemical reactions are as below:

$$2NH_2Cl + HOCl \rightleftharpoons N_2 + 3HCl + H_2O$$

$$2NH_2Cl + 2NHCl_2 + HOCl \rightleftharpoons N_2O + N_2 + 7HCl$$

The end products do not represent any residual chlorine. This fall in residual chlorine will continue with a further increase in chlorine dose and after a stage, the residual chlorine begins to increase in proportion to the added dose of chlorine. This point at which the free residual chlorine appears after the entire combined chlorine residual has been completely destroyed is referred to as breakpoint and the corresponding dosage is the breakpoint dosage. Breakpoint chlorination achieves the same results as super-chlorination in a rational manner and can therefore be construed as controlled super-chlorination in the case of polluted raw water. Figure 9.2 shows the breakpoint chlorination curve.

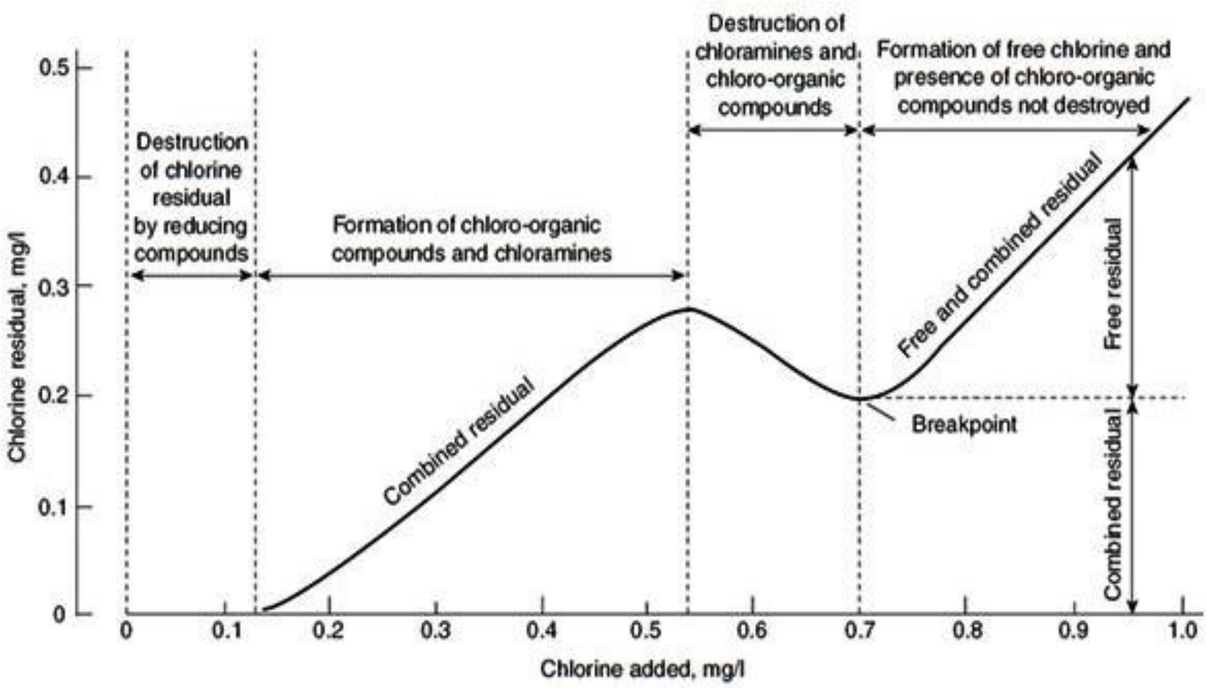


Figure 9.2: Break Point Chlorination Curve

9.6.4 Combined Available Residual Chlorination

This method involves the application of chlorine to the water to react with natural and added ammonia, to form a predominantly combined monochloramine chlorine residual. Doing this will produce the most reliable way to maintain the residual through part or all of a water treatment plant or distribution system. Chloramines are less effective disinfectants and oxidants than free available chlorine forms. But the residual will persist much longer than free available chlorine which has a tendency to diffuse and be lost. A minimum of 30 to 60 minutes of contact time must be provided before delivery to the consumer and in long transmission mains and non-looped distribution systems, monochloramine residual should be maintained to the furthest point in the system 24/7. Depending upon the characteristics of water, this can be accomplished as follows:

(i) Application of chlorine only, if sufficient ammonia is present in the water;

(ii) Addition of both chlorine and ammonia, if it contains little ammonia; or

(iii) Addition of ammonia, if free available residual chlorine is already present in water.

In order to control chlorine-ammonia treatment effectively, the optimum ratio of chlorine to ammonia has been found to be between 3:1 and 4:1 to optimise any remaining level of free ammonia in the system. Free ammonia in the system can cause nitrification in the distribution system and that should be avoided.

This practice is useful after filtration for controlling algae and bacterial growths, for reducing ‘red water’ troubles in distribution systems at dead ends, and for providing and maintaining a stable residual throughout the distribution system.

9.7 Real-Time Chlorine Concentration

The modern software available to the water sector pertaining to distribution system has the facility of predicting real-time chlorine concentration using the sensor data from distribution pipe network for 24×7 water supply system. The data generated by SCADA or the IoT sensors is pushed to the cloud service of digital twin technology, where this data is saved, and predictive analysis is made.

The data is then sent back to human machine interface (screen) of SCADA of ULB where the real-time graphs of chlorine concentration are delineated. Sensors have been discussed in Part B, Chapter 10: Automation of Water Supply of this manual.

9.8 Points of Chlorination

The use of chlorine at various stages of the water supply system right from raw water collection to the distribution network is a common practice and terms like pre-chlorination, post- chlorination and re-chlorination have come into common usage depending upon the points at which chlorine is applied.

9.8.1 Pre-chlorination

Pre-chlorination is the dosing of chlorine to raw water prior to any unit treatment process and is generally upstream of Parshall flume. The point of application, as well as dosage, will be determined by the objectives viz., control of biological growths in raw water conduits, promotion of improved coagulation, prevention of mud ball and slime formation in filters, reduction of taste, odour, and colour, and minimising the post-chlorination dosage when dealing with heavily polluted water. Pre-chlorination is also required to control and prevent the growth of algae. However, in order to reduce the formation of disinfection by-products (DBPs), the practice of dosing chlorine dioxide with chlorine is increasing. For the same reason, ozone application is increasingly used in pre-oxidation.

Satisfactory disinfection is obtained by Pre-chlorinating to maintain 0.10 to 0.20 mg/L free residual chlorine in the outlet of filter at normal pH values.

9.8.2 Post-chlorination

Post-chlorination is the application of chlorine to water before it enters the distribution system to maintain the required amount of free residual chlorine. Post-chlorination dose is applied with 30 minutes of contact time in chlorine contact tank before the water enters the clear water sump. Many times, a booster chlorine dose is required at the service reservoirs in the distribution system to maintain the residual chlorine level.

9.8.3 Re-chlorination

When the distribution system is long and complex, it may be difficult to maintain the minimum chlorine residual of 0.2 mg/L at the farthest end. To achieve this, if a very high dosage is applied at the post-chlorination stage, it would, apart from being costly, make the water unpalatable at the reaches close to the point of chlorination. The maintenance of the required residual, in such cases, can be accomplished by a stage-wise application of chlorine in the distribution system which is called re-chlorination. Re-chlorination is carried out in service reservoirs, booster pumping stations, or at points where the delivery mains supply to distribution zones. Real-time measurement for critical locations needs to be incorporated into the system.

9.8.4 Chlorine Residual

In the post-monsoon period, the minimum residual chlorine shall be maintained as 0.20 mg/L at the end of the distribution system, and during monsoon or in epidemics, 0.50 mg/L at the end of the distribution system. Practically, the chlorine concentration at entry point of distribution system or service reservoir and furthest point in the distribution is measured. This exercise is repeated for four to five readings and then a graph of initial chlorine concentration versus residual chlorine at end is plotted. From this graph, the required initial chlorine dose is determined.

Generally, groundwater sources are naturally pure because water percolates through various layers of soil. A required chlorine dose needs to be added in the service reservoir or the outlet of the tube well for direct pumping to maintain residual chlorine of 0.20 mg/L at end of the operation zone/DMA of distribution system.

Total chlorine and free residual chlorine can be determined using BIS Standard IS 3025 (Part 26): 2004, Methods of sampling and test (physical and chemical) for water and wastewater.

9.9 Application of Chlorine

Chlorine can be applied to water by three methods:

(i) By the addition of a weak solution prepared from bleaching powder, HTH (high-test hypochlorite), etc., for disinfecting small quantities of water.

(ii) By the addition of a weak solution of chlorine prepared by electrolysing a solution of brine.

(iii) By the addition of chlorine, either in gaseous form or in the form of a solution made by dissolving gaseous chlorine in a small auxiliary flow of water, the chlorine gas is obtained from pressurised chlorine cylinders.

The first method of chlorine application has the merits of simplicity, non-requirement of electrical energy, and relative safety in operation and handling as available chlorine is either in powder or solution form. However, the demerits include instability of bleaching powder, its hygroscopic nature, and a relatively low percentage of available chlorine (25%-33%). To overcome these disadvantages, some variants with the basic chemical compound of calcium hypochlorite are recommended. These compounds possess a high chlorine content of about 65%-70% and are stable, easily soluble, and non-hygroscopic. However, these are expensive and require safety in handling.

The second method of chlorine application requires the deployment of electro-chlorinators to prepare a chlorine solution from the electrolysis of water containing sodium chloride. An electro-chlorinator essentially comprises of a direct current (DC) source for providing energy for electrolysis, an electrode pair installed in a container, and a hypochlorite solution storage and dispensation device. During electrolysis, chlorine is not created as a gas but is available in solution form as a sodium hypochlorite solution. This is the major advantage of this technique as transportation, storage, and application of chlorine gas involve major safety considerations to avoid hazards and fatal accidents.

The third method of chlorine application is presently the common practice for medium to large public water supplies. However, it requires elaborate safety practices and the use of chlorinators or chlorine evaporators and auxiliary equipment.

9.10 Chlorinators

A chlorinator is a device designed for feeding chlorine to a water supply. Its functions are:

(a) To regulate the flow of gas from the chlorine container at the desired rate of flow.
(b) To indicate the flow rate of gas feeding.
(c) To provide means of properly mixing the gas either with an auxiliary supply of water or with the main body of the liquid to be disinfected.

The main components are summarised in Table 9.1.

Table 9.1: Chlorinator system components

Part	Purpose
Vacuum Regulator	Reduces the gas pressure from the container (minimum one bar) to the sub-atmospheric pressure of the chlorinator and

Part	Purpose
	adjusts the gas flow rate to correspond to the vacuum set by the adjustment of the V-notch plug within its orifice.
Pressure Relief System	Discharges chlorine gas to the outside through the pressure relief vent or valve, if excessive gas pressure in the chlorinator should occur.
Positioner	Controls the rate of gas flow through the chlorinator by adjusting the position of the V-notch plug within its orifice, generally by automatic control with a manual override.
Flowmeter	Indicates chlorinator feed rate. (Read the widest part or top of the float or centre of the ball for rate marked on tube).
Differential Regulating Valve	Ensures that the vacuum differential across the gas control V-notch plug is consistent.
Pressure Check Valve	Prevents water back feeding into the chlorinator from the injector.
Vacuum Relief System	Admits air into the chlorinator system through the vacuum relief vent or valve, if excessive vacuum should occur.
Pressure Gauges	Indicate gas pressure at the containers and water pressure at the injector.
Vacuum Gauges	Indicate vacuum in the chlorination system.
Injector	Creates the vacuum for the system and sucks the chlorine gas into the operating water supply to form the chlorine solution for injection into the water supply to be disinfected.
Vacuum Switch	A local or remote mounted vacuum switch provides an alarm in the event of a high or low vacuum condition signifying a loss of gas feed
Gas Warning Light, Audible Alarm and Air Blower Switch	Give warning that a predetermined level of chlorine gas has been detected in the air of the chlorine store and enables air blower to be switched on to displace gas from store via the low-level inlet and air duct to the outside.

9.10.1 Types of Feeders

Chlorinators are used for the control and measurement of chlorine in the gaseous state and to supply chlorine as a gas or an aqueous chlorine solution. The principle of operation of this equipment depends on the regulation of flow by establishing a pressure relationship between the upstream and downstream sides of either a constant or a variable orifice in the chlorine flow gas line. Control of the feed rate is affected either by varying the pressure differential across a fixed orifice (variable differential unit) or by varying the size of the orifice (constant differential unit). The requirement for chlorination equipment should adhere to IS 10553 (Part 1) (1983, reaffirmed 2022).

These feeders are of two types, viz., (a) Pressure type and (b) Vacuum type.

(a) Pressure Type Gravity Feed Chlorinator (IS 10553-4, 1983, Reaffirmed 2021)

(b) Vacuum Type Chlorinator (IS 10553-2, 1983, Reaffirmed 2021)

9.10.2 Number of Chlorine Cylinders or Containers

The normal chlorine dosage required to disinfect water in public water supply systems, not subject to significant pollution, would not exceed 2 mg/L. The actual chlorine dosage has to be determined on the basis of chlorine demand tests. The chlorine feed rate is then computed by dividing the expected maximum dosage of chlorine by the maximum flow rate.

Total daily chlorine requirements can be estimated from the daily average consumption in a maximum day. The peak and the minimum rate requirements should be taken into consideration when designing a chlorine supply and feeder system and not merely the total daily requirements of chlorine.

When chlorine gas is withdrawn from a cylinder containing the liquefied gas, the pressure drops, and the liquid 'boils' liberating more gas till the pressure is restored. This boiling absorbs heat continuously, then produces a cooling effect in the liquid region. If the withdrawal is continued, the liquid may freeze, and no more gas will be evolved. It is, therefore, essential to keep the atmosphere, around the containers in service, warm and to ensure that there is no abnormal rate of withdrawal from a single container with heavy demand for gas.

The recommended discharge rates are approximately 6.5 to 7.5 kg/hr. from a one-tonne container and 0.8 kg/hr. from cylinders. Equipment should have sufficient capacity to exceed the highest expected demand at any time and to provide continuous effective discharge under all prevailing hydraulic conditions. It is a good practice to arrange for duplicate/standby equipment since the disinfection process cannot be stopped at any time.

When the gas discharge rate from a single container will not meet the requirements, two or more can be connected to a manifold and discharge simultaneously. It is advisable not to couple more than four containers to a manifold. When discharging through a manifold, care must be taken that all the containers are at the same temperature, particularly when connecting a new cylinder to the manifold. Where more than three or four cylinders are used, the connections would be arranged in groups so that one complete group can be changed at a time. Storage of chlorine lasting a month or two should be provided. It is advisable to keep the full cylinders in the same room as the cylinders in service.

The chlorine tonner/cylinder are available according to their weights in 45, 67, 100 and 900 kg. For using this type of tonners, the rule of thumb is 1 MLD capacity plant requires 1 kg of chlorine to generate 1 mg/L of residual chlorine. The tonner/cylinder replacement period is normally considered as 45 days.

9.10.3 Chlorine Cylinder/Tonner Store and Chlorination Room

The chlorine cylinders and feeders should be housed in an isolated room, easily accessible, close to the point of application, and convenient for truck loading and safe container handling. The floor should be at least 15 cm above the surrounding ground and drainage, should have at least two exit doors or the building should have at least two exit doors for cross ventilation that allows an approximate air change in 10 minutes. For small installations, the provision of a ventilator opening at the bottom, one opposite the other is adequate.

Separate and reasonably gas-tight enclosures, opening to the outdoors, should be provided for housing the chlorine feeding equipment in large installations and buildings occupied by waterworks personnel. These enclosures should be vented to the upper atmosphere and equipped with positive means of exhaust (near the floor level, at the centre of the room or opposite to the entrance) capable of a complete air change within 2 to 4 minutes in an emergency. A satisfactory ventilation scheme involves a combination of fresh air and an exhaust system, consisting of fans that force the fresh air into the enclosure through openings near the ceiling with exhaust fans to clear away any chlorine contaminated air near floor level. The design of the exhaust system should not include the natural ventilation that may be available.

Additionally, a gas scrubber system should also be provided along with an absorption tower for safety.

9.10.4 Chlorine Evaporators

Chlorine vaporisers, better known as evaporators, are needed whenever conditions require the withdrawal of liquid chlorine from the containers. Typically, this action is necessary when the daily chlorine requirements exceed 100 to 1000 kg and would require the manifold provisioning for the excessive number of containers. Evaporators provide the heat necessary to vaporise or change the liquid chlorine to the gaseous state so that it may be handled in the normal fashion by the other components of the chlorination system.

An evaporator consists of a chlorine pressure vessel to which heat can be applied under controlled conditions, the source of heat may be electricity, steam, or hot water with several different models and versions available from different manufacturers. The most commonly used models are electrically heated and have maximum rated capacities of 150 kg/hr. In all cases, the source of heat is thermostatically controlled to maintain a constant temperature and ensure a superheated chlorine gas output.

The liquid absorbs heat from the water chamber through the wall of the pressure vessel until it reaches the vaporisation temperature and boils, releasing chlorine gas. With the chlorination system in operation, gas is withdrawn from the pressure vessel, allowing more liquid to enter the system and continue the process. The gas leaving the vessel contacts the hot upper wall and causes the gas to be heated to a higher or superheated temperature. Baffles may be used to assist heat transfer.

As heat is absorbed by the chlorine, the water is cooled until it drops below the control thermostat setting and causes the electric immersion heater to be actuated. The heater remains on until the water reaches the upper limit of the thermostat. At this point, the heater is shut off and this cycle is continued as long as the evaporator is in use. As chlorine is vapourised, any impurities contained in the liquid are left behind, coating the inside surface of the pressure vessel. This coating acts as an insulator that inhibits heat transfer from the water to the chlorine and necessitates higher and higher water bath temperature settings. Eventually, the pressure vessel will require cleaning as the evaporator will be unable to vaporise the desired amount of chlorine gas at the desired degree of superheat. The frequency of cleaning will vary from installation to installation and is a function of chlorine purity and the chlorine feed rate. The evaporator manufacturer's cleaning instructions should be followed closely for best results.

Typically, evaporators are supplied with various gauges and controls, in addition to the control thermostat, to permit simple, safe operation. Taken individually, they include the following components.

- Control thermostat senses water temperature and turns the heater on and off to control water bath temperature.
- Water-level gauge is a sight glass that permits the operator to observe the water bath level.
- A chlorine bath-pressure gauge indicates chlorine gas pressure within the pressure vessel.
- The chlorine gas temperature gauge indicates the temperature of the superheated chlorine gas leaving the pressure vessel.

- Water temperature gauge indicates the temperature of the water bath.
- Low water temperature switch actuates a persistent lower water bath temperature, which normally indicates loss of the heater system. The switch may be used to actuate an alarm and automatically close the chlorine gas ‘shut off’ valve to avoid the pulling of liquid chlorine into the gas system.
- High water temperature switch actuates a persistent high water bath temperature. Excessive temperature may be caused i) by failure of the heater controls, which allows the heater to remain on; or ii) by loss of water from the water bath. The switch may be used to actuate an alarm and turn off power to the electric heater.
- Low water level switch actuates a persistent low water bath level, caused by a failure of the water supply system. Switches may be used to actuate an alarm and automatically open a water valve.
- Magnetic heater contactor acts in response to the control thermostat to energise or de-energise the electric immersion heater.
- Cathodic protection protects from corrosion to all metal surfaces of the evaporator in contact with water.
- The vent alarm switch actuates whenever gas flow occurs in the vent line and indicates that the gas pressure relief valve is opened.
- A liquid chlorine expansion tank protects the liquid chlorine piping system from damage due to overpressure by offering a chamber into which the liquid can expand. This tank is strongly recommended for installation in any section of liquid piping which may be purposely or accidentally isolated by closing valves.

Some, or all, of the aforementioned controls, gauges, and accessories may be supplied with an evaporator.

9.11 Electrolytic Chlorinators or On-site Chlorine Generators

Rather than storing traditional chlorine gas chemicals such as sodium or calcium hypochlorite, one can use electricity, salt, and water to generate sodium hypochlorite, safely, and on demand. This technology is commonly referred to as electro-chlorination (EC), chlorine electrolysis, or in-situ chlorine generation.

EC system works on a simple principle of electrolysis of brine solution (sodium chloride salt solution). There are two electrodes - anode and cathode, placed at a specific distance, between which the brine solution is passed. A low voltage is applied. The potential difference is responsible to create a current through the brine, which is a good conductor of electricity. In this process, dissociated salt (NaCl) breaks down into Na+ and Cl- ions. A small amount of hydrogen gas is also evolved. Following reactions take place:

$$NaCl + H_2O + 2e^- = NaClO + H_2$$

NaClO (or NaOCl) is sodium hypochlorite, which is then injected for disinfection at a suitable point.

Hydrogen gas, being in very small amounts, does not impose any operational hazard. It is suitably vented off.

A representative diagram of EC system is given in Figure 9.3:

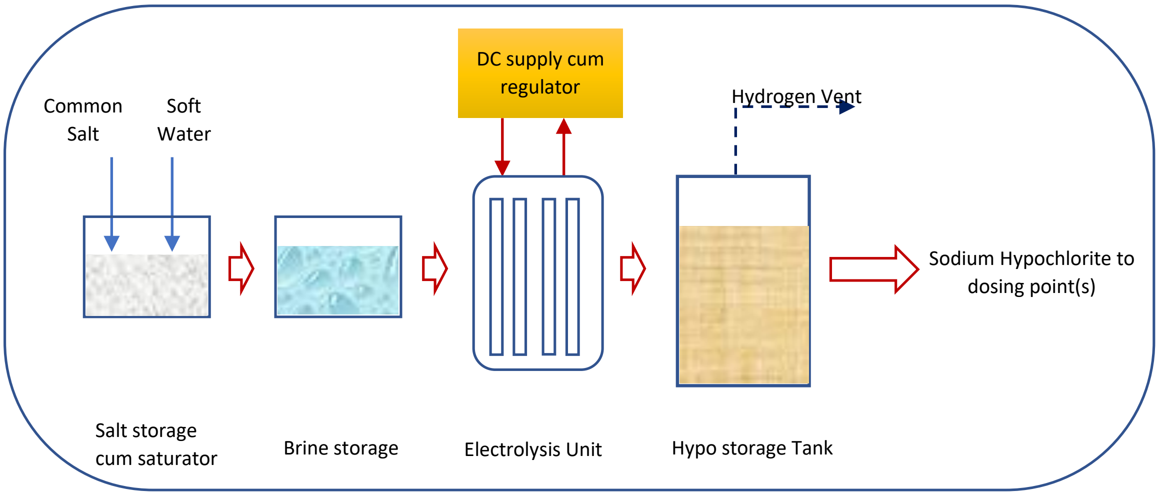


Figure 9.3: Electro-chlorination (EC) system

The hydrogen by-product is diluted with a large volume of air in an enclosed ventilation system and is discharged safely into the atmosphere. The supersaturated salt solution is diluted to a 3% concentration. Chlorine is generated at the anode and caustic soda is generated at the cathode. They react with each other to form dilute (0.5% to 1%) sodium hypochlorite solution. Electro-chlorinators are commercially available in India in capacities ranging up to 1000 g/hr. of chlorine using the salt solution and up to 100 kg/hr. of available chlorine using seawater. It is reported that power consumption may be less than 5 kWh/kg of available chlorine for units with capacities greater than 500 g/hr. and common salt requirements are of the order of 4-4.5 kg/hr. of available chlorine. However, no IS specifications are presently available for these electro-chlorinators, and these electro-chlorinators are based on emerging technology.

Though power consumption is low, and cost of common salt is negligible, it is experienced that the capital cost of these units is huge and require skilled supervision. Also, annual maintenance charges are on higher side.

9.12 Ancillary Equipment

9.12.1 Weighing Machines

Weighing scales are necessary to record the weight of chlorine used in 24 hours which would serve as a check of the daily consumption and also enable the cylinders to be changed when they get empty.

9.12.2 Personnel Protection Equipment

Severe exposure and potential health hazards exist wherever toxic compressed gas and other respiratory irritants are handled or used. An approved gas mask should be provided for every employee involved with handling them. Additionally, suitable protective equipment for emergency use should be available outside rooms where hazardous materials are located near the entrance, away from areas of likely contamination. Such equipment might be provided in several locations at larger installations.

Canister type gas masks with a full-face piece and specific or all-purpose commodity canister should be used only for relatively short exposure periods and only if it has been clearly established that sufficient oxygen is present (not less than 16% in air) and that contamination does not exceed the allowable level (1% of chlorine). Canister masks might not be useful in emergencies since these criteria might not be readily ascertained, especially if suitable forced ventilation schemes are not provided. Regular replacement of overage canisters, even though unused, is recommended.

Self-contained breathing apparatus, with a full-face piece and a cylinder of air or oxygen carried on the body or with a canister that produces oxygen chemically, is suitable for high contaminant concentrations and is the preferred means of respiratory protection. Protection is provided for a period that varies with the amount of air, oxygen, or oxygen-producing chemicals carried.

Respiratory protective equipment should be carefully maintained, inspected, and cleaned after each use and at regular intervals. Defective or inoperable equipment is worse than none at all. All such equipment should be used and maintained in strict accord with the manufacturer's instructions, no person should enter contaminated areas unless attended by an observer who can rescue him in the event of a respiratory problem or other emergencies.

It is a good practice to provide eye protection devices (or masks with full-face pieces) and other protective clothing for workers exposed to hazardous materials. Emergency showers, eye baths, or other suitable water-flush systems should be provided in convenient locations for use by accidentally exposed personnel. Installation of an automatic chlorine leak detector with or without visible or audible alarms should be considered. The typical arrangement of PPE at the time of installation is shown in Figure 9.4.

Figure 9.4: Typical arrangement of PPE at time of Installation

9.12.3 Chlorine Detectors

Continuous monitoring of the atmosphere in areas where chlorine is stored and fed is an important aspect of any safety programme. Instruments for this purpose are called chlorine detectors, which are not to be confused with the detectors used for measuring residual chlorine in the water.

Concentrations are expressed as ppm by volume in air, not as parts per million parts by weight, as the expression is used to denote concentrations in water. For comparison, 1 ppm of chlorine by volume in air is equivalent to 3 mg/m^3 of air. The threshold of odour perception is about 3 ppm.

Two types of detectors are available. In type one, the air to be sampled is directed to a rotating drum covered with a strip of sensitised paper. The paper is white and light is reflected from it to a photoelectric cell. The current from the cell is amplified and used to keep an electric relay open in an alarm circuit. If the air sample contains chlorine, the paper darkens, the light is absorbed, the current from the photoelectric cell drops below that required to keep the relay open and thus the alarm circuit is energised.

In the second type, air from the point or points of sampling is drawn to the detector by an air pump through a filter and flowmeter that indicates the sample flow rate. The air sample is directed to an electrochemical sensing cell, the electric output of which increases with the presence of chlorine. A meter movement is incorporated to indicate visually the strength of the chlorine in the air and an adjustable switch is included to provide a contact closure for remote audible/visual alarms and, or remotely operate an exhaust fan.

9.12.4 Automatic Changeover System

Increased emphasis on the need for uninterrupted chlorination has led to the use of automatic changeover systems, particularly at unmanned stations. The basic concept of these systems is to switch from a depleted source of chlorine to a standby source automatically without the presence of an operator. Several methods have been used to accomplish this. One system consists of electrically operated chlorine shut-off valves, actuated by a chlorine pressure switch that senses the loss of chlorine pressure due to empty cylinders. Another system uses two pressure-reducing valves, each attached to its own source of chlorine and manifold on the downstream sides. The pressure settings of the two valves are adjusted so that the valves control at a pressure approximately 3.52 m H_2O apart. Since a pressure-reducing valve will not open until the downstream pressure is lower than its setting, the valve with the higher setting opens first, allowing gas to flow through the valve from its source. This process continues until the first source is depleted and the downstream pressure drops to the setting of the second valve, at which point it opens and chlorine flows from the standby source.

The recent development of small cylinder-mounted chlorinators has added more types of automatic changeover systems to the marketplace. It is not necessary to detail the operation of each, but merely to state that they meet the basic need of permitting continuous chlorine feed in a simple inexpensive manner for even the smallest gas chlorination facilities.

9.13 Safety Considerations

(i) Only trained personnel should be permitted to handle chlorine cylinders and chlorinating equipment. They should be made aware of the hazards involved, the precautions to be observed and first aid to be rendered in emergencies. Rubber gloves, aprons, and suitable gas masks should be provided. These should be housed in an easily accessible (unlocked) cupboard placed outside the chlorinator room. It is very important that the operating personnel are trained in the proper use of gas masks. A faulty gas mask is worse than none at all. Hence, it is very important that these are tested frequently, and the containers are changed at proper intervals.

(ii) When a chlorine leak occurs, the mechanical ventilation system should be opened immediately before any person enters the chlorine room. It must be made a point that chlorine container valves are closed first before any investigation is started.

(iii) Cylinders containing chlorine should be handled gently. They should not be bumped, dropped, or rolled on the ground and no object should be allowed to strike them with force. The protective hoods over the valve should always be kept in place except when the cylinders are in use. Flames should never be applied to chlorine cylinders or their valves.

(iv) Cylinders should not be stored in the open or damp places. Empty cylinders should be stored away from full cylinders so that they do not get mixed up. It would be desirable to tag the empties as an additional precaution. Incidentally, this will ensure the prompt return of used cylinders.

(v) In case the valve is found to be stuck, the cylinder should be immediately returned to the supplier. No attempt should be made to ease a stuck valve by hammer, as this is very dangerous.

(vi) Only the spanners prescribed for use should be used as it is important not to put too much leverage on the valves.

(vii) Cylinders, as well as the chlorinators, must be tested at the start and end of every shift period, for leaks, first by trying to detect the sharp irritating smell of chlorine, then by passing over each cylinder and around each valve and pipe connections, a rod with a small cotton wool swab tied on the end, dipped in an aqueous solution of ammonia. Any leakage noticed anywhere must be attended immediately otherwise same is going to lead major trouble in the plant. If chlorine is present in the air, the swab will appear to 'smoke' due to the formation of white clouds of ammonium chloride. If the leak appears to be heavy, all persons not directly concerned should leave the area and the operator should put on his mask and make a thorough search for the leak. In tracing a leak, always work ‘downstream’, i.e., start at the cylinder and work down along the line of flow until the leak is found. It will save many valuable minutes over the practice of starting in the middle of the chlorinator and searching vaguely back and forth over the whole equipment.

(viii) Water should never be applied to a chlorine leak to stop it as it will only make the release of ‘free chlorine’. If the leak is in the chlorinator, the cylinder should be immediately shut off until the pressure has reduced. The joint or gasket should be repaired, and replaced with new packing, if necessary.

(ix) Solvents such as petroleum, hydrocarbons, or alcohols should not be used for cleaning parts that come in contact with chlorine. The safe solvents are chloroform and carbon tetrachloride. Grease should never be used where it can come in contact with chlorine as it forms a voluminous frothy substance on reaction with chlorine. The only special type of cement, recommended by manufacturers, should be used.

(x) No direct flame should be applied to a chlorine cylinder, when heating becomes necessary, as this is hazardous. A water bath, with a controlled temperature not to exceed 27 °C, should be used.

(xi) Before disconnecting the flexible lids from containers to gas headers, the cylinder valves should be closed first and then the gas under pressure should be drawn from the header and flexible lids before the header valve is closed. The exhaust system should be turned on and operated on while the cylinders are being disconnected or repairs are being made.

The arrangement inside of chlorinator room and tonner room at the site is shown in Figure 9.5. The new development in safety for chlorine tonner as tonner safety container shells is shown in Figure 9.6. The arrangement at the site from outside of a tonner room is shown in Figures 9.7 and Figure 9.8.

Figure 9.5: Chlorinators (Vacuum Type), booster pumps, Tonner handling in Maharashtra

Figure 9.6: New Development as tonner safety container shells have been developed

Figure 9.7: Installation of chlorine cylinders (100 kg) and chlorinators At Packaged water treatment plant, Rajapur, Dist. Ahemadnagar, Maharashtra

Figure 9.8: Chlorination Room and tonner yard at Ghulewadi (10 MLD) WTP, Dist. Ahemadnagar, Maharashtra

Other safety factors (IS 4263, 1967: Code of Safety for Chlorine, Reaffirmed 2018)

1. Initial lead pipe (armoured plastic or copper) shall be replaced after six months which is normally practised for all types of chlorinators.
2. The gas valve can be opened and closed as per requirement, but the water flow system shall be continuous in operation. It will ensure the remaining gas gets into the solution. This safety

is not seen in any other chlorinators. Hence, there is no O&M to the system. Flowing water is not wasted, it is let in the sump.

3. The tonner/cylinder valve joint and the initial pipe joint with the unit shall be carefully tested with ammonium chloride for checking the leakage of gas.
4. Green plants shall be kept in the vicinity of tonner/cylinders to identify the change of colour of leaves. This practice is for using any type of chlorinators.
5. For safety against tonner/cylinder leakage arrangement to move it to CaO solution tanks shall be provided. This is also common care to be taken for all types of chlorinators.

9.13.1 Handling Emergencies

As soon as there is an indication of a chlorine leak or other abnormal condition, corrective steps should be taken. Leaks never get better by themselves; they always get worse if not promptly and suitably repaired. Authorised trained personnel with suitable gas masks should investigate and all other persons should be kept away from the affected area. The ventilation system should be placed in operation immediately. Unconfined chlorine, being heavier than air, tends to lie close to ground levels (the characteristic must be kept in mind in designing a chemical storage and use areas and appropriate natural or mechanical ventilation system). If leaks cannot be handled promptly, the chemical supplier or nearest office or plant of the producer should be called immediately for emergency assistance.

In case of fire, containers should be removed from the fire zone immediately. Portable tanks, tank cars, trucks, and barges should be disconnected and if possible, should be removed from the fire zone. Even if there are no visible leaks, water should never be applied to a chlorine container. Chlorine is only slightly soluble in water and the corrosive character of its reaction with water always will intensify the leak. In addition, the heat supplied by even cold water will increase the vaporisation rate. Leaking chlorine containers similarly should not be thrown into a body of water because the leak will be aggravated and the container might float when still partially full, allowing uncontrolled gas evolution at the surface.

If a leak occurs in equipment or piping, the supply should be discontinued and the material under pressure at the leak should be disposed of. Leaks around the container valve stem usually can be stopped by tightening the pack out or gland. If this action does not stop the leak, the container valve should be closed and material under pressure in the outlet piping should be disposed of. If the valve does not shut off tight, the outlet plug, or cap should be applied. In the case of a leaking valve of a tonne container, the container should be positioned so that the valves are in a vertical plane with the leaky valve on the top. Additionally, following actions can be taken if a tonner is found to be leaking:

- Position cylinders or tonne containers so that gas instead of liquid escapes. The containers may be insulated with sacks, earth, etc., to decrease the absorption of heat and the discharge rate.
- Apply appropriate emergency capping devices, if available.
- Call the supplier or nearest producer for emergency assistance.
- If practical, reduce pressure in the container by removing the gas to process or suitable disposal system. Caustic soda, soda ash, or other suitable alkali absorption system should be provided for disposing of chlorine from leaking cylinders and tonne containers.
- In some cases, it might be desirable and possible to move the container to an isolated spot where it will do the least harm.

Safety in handling hazardous materials depends to a great extent upon the effectiveness of employee education, proper safety instructions, intelligent supervision, and the use of safe equipment. Training for both new and old employees should be conducted periodically to maintain a high degree of safety in handling procedures. Employees should be thoroughly informed of the hazards that may result from improper handling. They should be cautioned to prevent leaks and thoroughly instructed regarding proper action to take in case leaks do occur. Each employee should know what to do in an emergency and should be fully informed about first aid measures. In addition, employee training should encompass the following:

- Instruction and periodic drills or quizzes regarding the locations, purpose, and use of emergency fire-fighting equipment, alarms, and emergency crash shutdown equipment such as valves and switches.
- Instruction and periodic drills or quizzes regarding the locations, purpose, and use of personnel protective equipment.
- Instruction and periodic drills or quizzes regarding the locations, purpose, and use of safety showers, eye baths, bubblier drinking fountains, and the closest source of water for use in emergencies.
- Instruction and periodic drills or quizzes of selected employees regarding the locations, purpose, and use of respiratory first-aid equipment.
- Instruction to avoid inhalation of toxic vapours and all direct contact with corrosive liquids.
- Instruction to report to the proper authority all leaks and equipment failures.

Gas Scrubber and/or neutralisation tank shall also be provisioned in the chlorine storage room. These are discussed in the following sections.

9.13.2 Gas Scrubber

This system consists of a blower, an alkali (NaOH) tank, an absorption tower packed with Raschig rings, an alkali circulation pump, piping valves, lightweight FRP, and PVC duct. In the event of a leak that is uncontrollable with the emergency kit, this system would allow the person to breathe easily rather than panic. In this system, the leak is confined by a hood covering the leaking container, sucking the chlorine by blower and delivering it to the absorption tower (Figure 9.9). The chlorine leak absorption capacity of the system is kept 100 kg/hr. and 200 kg/hr. for 100 kg. cylinder and 900 kg. tonner respectively. If one is confronted with other container leaks, one or more of the following procedures should be considered:

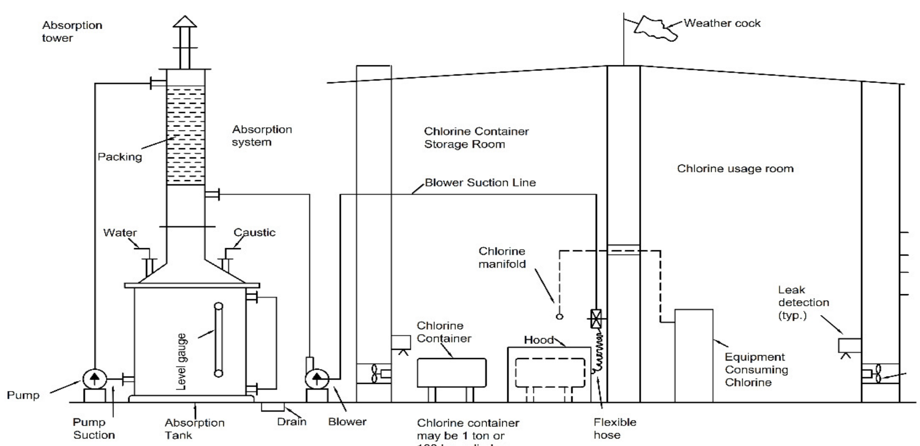


Figure 9.9: Typical Chlorine Leak Absorption System

9.13.3 Neutralisation tank

As a regular part of chlorine storage and use, provisions shall be made for emergency disposal of chlorine from leaking cylinders or ton-containers. Chlorine may be absorbed in solutions of caustic soda or soda ash or agitated hydrated lime slurries. Caustic soda is recommended as it absorbs chlorine more readily. The proportions of alkali and water recommended for this purpose are given below in Table 9.2:

Table 9.2: Quantities of Lime and Soda Ash for emergency disposal of chlorine

Chlorine Container Capacity	Soda Ash and Water		Hydrated Lime and Water	
kg	Weight (kg)	Volume (l)	Weight (kg)	Volume (l)
45	136	450	58	566
68	220	680	82	815
900	2,720	9,050	1,160	11,350
Source: Section 8.2 in IS 4263				

9.14 Chlorine Compounds

Chlorine may also be applied in the form of compounds such as bleaching powder or calcium or sodium hypochlorite which make the chlorine available when they come into contact with water. These are used for disinfection of small water supplies having capacities up to 1 MLD as a primary disinfectant and as an emergency disinfectant for large plants.

9.14.1 Bleaching Powder (IS 1065: Part 2, 2019)

Bleaching powder ($CaOCl_2$) is a variable mixture of calcium hydroxide, calcium chloride, and calcium hypochlorite. When it is mixed with water, the calcium hypochlorite decomposes into calcium chloride and chlorine. The action exerted by bleaching powder, is, therefore, similar to that of gaseous chlorine in the water. Bleaching powder is characterised by its content of available chlorine, i.e., the chlorine which can be liberated by complete reaction with water. Commercial brands have available chlorine of 20% to 30%, i.e., 20 to 30 parts by weight of chlorine per 100 parts by weight of bleaching powder. Since fresh bleaching powder contains only 20%-30% available chlorine, its use involves the extra expense of transporting and storing the inert material. Furthermore, bleaching powder is an unstable compound and loses its available chlorine on storage. Bleaching powder is hygroscopic in nature, it loses its chlorine strength rapidly due to storage and, hence, should not be stored for more than three months. Hence, a proper storage arrangement to keep the bleaching powder/bags dry is important. The chlorine content should be periodically measured and recorded to decide the required dosage for disinfection.

Bleaching powder is generally made into a thin slurry with water, and the supernatant, which contains the chlorine in the solution, is applied to the water by a suitable feeding mechanism such as a float-operated gravity box. In every installation, the solution may be applied through a drip feed mechanism. Devices that can give constant feed can be easily fabricated. An injector may be fitted on a bleed line on the pump discharge to suck the solution of the powder in proportion to the flow of water.

All these considerations make its use uneconomical except in very small installations or for special cases such as disinfection of mains. Figure 9.10 show an arrangement at the site for dosing of bleaching powder.

Figure 9.10: Bleaching Powder dosing tanks at 10 MLD Vaijapur WTP, Maharashtra

9.14.2 Hypochlorites

The chemicals used are sodium hypochlorite and calcium hypochlorite. Specially fortified brands of calcium hypochlorite, such as perchloron and HTH, can have 60-70 per cent available chlorine. Calcium hypochlorite can be fed either in the dry or solution form, while sodium hypochlorite is fed as a solution. The solution form is usually preferred. Corrosion-resistant materials such as ceramics, glass, plastic, or special rubber should be used while handling hypochlorite solutions. Generally, 1% to 2% chlorine solutions are prepared and fed directly through solution feeders. Usually, constant head gravity devices with adjustable orifices are used to dose chlorine solution in the tanks. These can be fed through chemical proportioning pumps and can be injected under pressure into pressure pipelines by venturi or orifice feeders.

Sodium Hypochlorite

Sodium hypochlorite (NaOCl) is a solution made from reacting chlorine with a sodium hydroxide solution. These two reactants are the major co-products from most chlor-alkali cells. Sodium hypochlorite, commonly referred to as bleach, has a variety of uses and is an excellent disinfectant/antimicrobial agent. It is a broad-spectrum disinfectant that is effective for the disinfection of viruses and bacteria. Sodium hypochlorite is most often encountered as a pale greenish-yellow dilute solution referred to as liquid bleach, which is a household chemical widely used (since the 18th century) as a disinfectant or a bleaching agent. In solution, the compound is unstable and easily decomposes, liberating chlorine which is the active principal component of such products. The pH of hypochlorite solutions should be raised to over 11 in order to extend the shelf life before it is used.

Sodium hypochlorite can be produced in two ways. One is by dissolving salt in softened water, resulting in a concentrated brine. This brine is then electrolysed to form a sodium hypochlorite solution containing 150 grams of active chlorine per litre. During this reaction, hydrogen gas is also formed. The chemical also can be produced by adding chlorine gas to caustic soda, producing sodium hypochlorite, water and salt as described earlier as electro-chlorinator.

In drinking water facilities, 12 per cent sodium hypochlorite is a disinfectant. In the time that it takes to ship the chemical, it typically degrades to 11 per cent. In the calculations of dosages or concentrations, 10 per cent should be used as the starting point. The decline is predictable if environmental factors are controlled.

Sodium hypochlorite is a strong oxidiser. Oxidation reactions are corrosive, and solutions can burn skin on contact. The strength of commercially available sodium hypochlorite is 10%-15%. Available chlorine is 12%-15%. The shelf life of the solution is six months. Both calcium and sodium hypochlorite are non-hazardous in nature but requires safe handling procedures such as protective gear and eye protection.

Calcium hypochlorite

Calcium hypochlorite [$Ca(OCl_2)$] is an inorganic compound. It has a high alkaline pH level of around 11-12 and contains 25%-30% free available chlorine. It is the main active ingredient of commercial products called bleaching powder, chlorine powder, or chlorinated lime, used for water treatment and as a bleaching agent. This is described in earlier section as bleaching powder.

9.14.3 ClO_2

Chlorine dioxide is an unstable gas, a free radical, and exists as a greenish-yellow to orange gas at room temperature with a characteristic pungent chlorine-like odour. Figure 9.11 show a schematic process diagram and typical arrangement for ClO_2. It is formed by reacting a strong solution of chlorine (7,500 mg/L of Cl_2 at pH 3.5) with sodium chlorite.

$$Cl_2 + 2NaClO_2 \rightleftharpoons 2ClO_2 + 2NaCl$$

The theoretical ratio of chlorine to sodium chlorite is 1:2.6. In practice, however, a large excess of chlorine is generally applied in order to avoid unreacted chlorite in the treated water. Chlorine dioxide is unstable and subject to explosion in gaseous form and easily be detonated by sunlight or heat, but aqueous solutions of the gas are stable and safe. Its solubility in water is 3 g/L at 20 °C.

Chlorine dioxide has many different types of uses, particularly in water treatment. Among these are disinfection, bleaching, and chemical oxidation. It has been reported to be a good bactericide and algaecide, and its bactericidal efficiency are relatively unaffected by pH between 6 and 10. It does not combine with ammonia and most organic impurities before oxidising them. The common dosages of chlorine dioxide range from 0.2 to 0.3 mg/L. Although chlorine dioxide is itself a disinfectant, the excess chlorine used in its generation, apart from ClO_2 is generally counted upon to achieve disinfection. It can be effectively used for the destruction of tastes and odours, particularly those which are caused by phenolic substances. ClO_2 does not produce trihalomethane or HAAS, which are the precursor to the formation of DPBs. There is a growing worldwide practice to use a combination of chlorine dioxide along with chlorine in the pre-chlorination units.

Gas phase chlorine dioxide concentrations in excess of 10% can decompose rapidly. This is the reason that chlorine dioxide must be generated at its point of use. For on-site generators, three feed chemical combinations are available: 1) chlorine-sodium chlorite, 2) acid-sodium hypochlorite-sodium chlorite, and 3) acid-sodium chlorite. In India, in some plants in the public water sector, ClO2 and combination with NaClO have shown promising results to treat algal laden water and vegetation imparted colour.

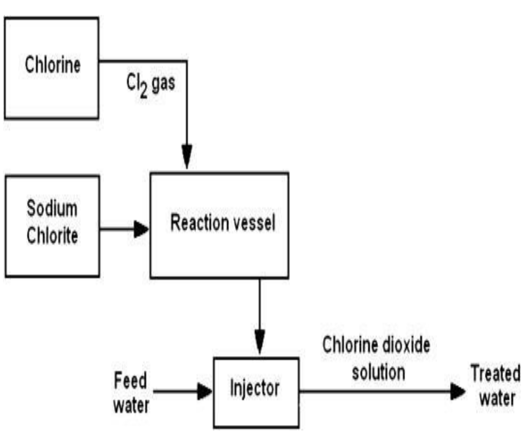


Figure 9.11: Schematic process Diagram and typical arrangement

9.14.4 Sodium dichloroisocyanurate

Sodium dichloroisocyanurate is the sodium salt of a chlorinated hydroxytriazine and is used as a source of free available chlorine, in the form of hypochlorous acid, for the disinfection of water. It is widely used as a stable source of chlorine for the disinfection of swimming pools and in the food industry. It is also used as a means of disinfecting drinking water, primarily in emergencies, when it provides an easy-to-use source of free chlorine, and, more recently, as the form of chlorine for household point-of-use water treatment. The BIS Standard sodium dichloroisocyanurate, IS 15733:2008, Reaffirmed 2019, should be followed when using it for disinfection purposes.

9.15 Chlorine Contact Tanks (For Post-Chlorination)

The effectiveness of chlorination depends on the bacteriological and virus load, chlorine concentration, contact time, and temperature. It is commonly seen that chlorine or its compound are normally dosed in treated water channel of filter house or treated water sump or treated water reservoir. It is not possible to dose chlorine uniformly using such methods. Hence, it is recommended to provide a dedicated chlorine contact tank/chamber in the treated water sump/clear water reservoir. In the existing plants, they can be accommodated in between the treated water channel of the filter house and the treated water sump. As a practice, the detention time of the chlorine contact tank is recommended as 30 min. It shall be constructed in RCC with a roof slab in square, rectangular, or circular shape. The tank shall have baffles so that the water flows horizontally. At the inlet to the tank, a feeder pipe (perforated) shall be provided to distribute the chlorine solution up to the entire depth of the tank. The outlet to the tank shall be at the top (Figure 9.12 and Figure 9.13). The construction of the tank should be in RCC M30 or above and the plaster (CM 1:2) should be at least 0.40 m in thickness. Other commercially developed products like FRP or paints which resist chlorine corrosion should preferably be applied. The inlet of chlorine contact tank (CCT) should be at the bottom, and outlet at the top. CCT should be in two compartments for plant of capacity more than 100 MLD with separate inlet and outlet for each compartment.

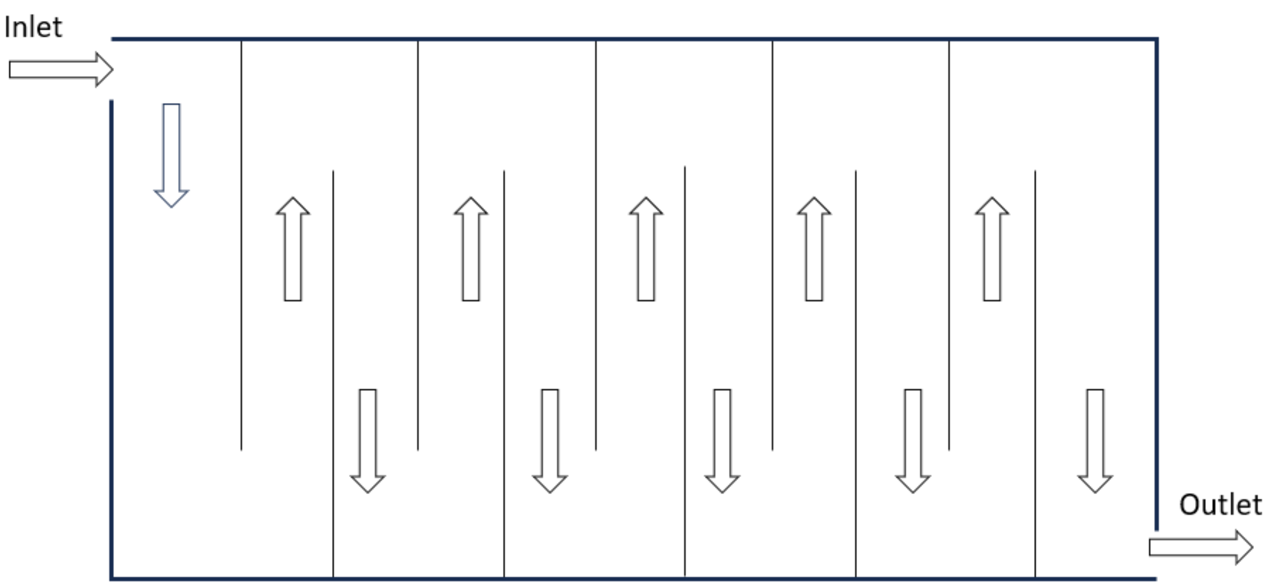


Figure 9.12: Schematic (Plan) Rectangular basin with baffle walls

Figure 9.13: Circular contact tank under construction

9.16 Disinfection Methods other than Chlorination

Chlorine and its compounds are most widely used for disinfection. However, possible formation of carcinogenic by-products, by reaction of chlorine with some organics and change in tastes and odours due to the reactions of chlorine with some water constituents, have been reported.

Various other agents of disinfection are available and some of them such as ozone and ultraviolet rays, are finding increasing usage in water treatment practice. Broadly, the three main types of disinfectants are (1) physical agents including heat, (2) chemical agents, and (3) radiations of various types such as ultraviolet rays, gamma rays, and x-rays. Some of these disinfectants are discussed in subsequent sections.

9.16.1 Heat

The boiling of water will disinfect it. This practice, however, cannot be used to disinfect municipal supplies for economic reasons. Boiling of water is applicable for disinfecting an individual's drinking water in emergencies like accidental contamination of public water supply or during an epidemic breakout. The thermal resistance of different microorganisms and viruses varies significantly with spores being up to 3,000,000 times more resistant than *E. coli*, viruses, and bacteriophages. As no important water-borne disease is caused by spore-forming bacteria or other heat-resistant organisms, boiling of water can render the water safe for drinking purposes against diseases. A Continuous flow water pasteurises with flow rates of 1000 lph, and is available for use.

9.16.2 Chemical Disinfectants

Chemical disinfectants are commonly grouped under the following categories:

(i) Oxidising chemicals include halogens, ozone, and other oxidants such as potassium permanganate and hydrogen peroxide.

(ii) Metal ions.

(iii) Alkalis and acids.

(iv) Surface active chemicals.

9.16.3 Halogens other than Chlorine

Halogens are oxidising agents and include fluorine being the strongest and iodine the weakest oxidising agent. However, disinfecting efficiency does not correlate directly with an oxidising capacity of a disinfectant. As fluorine can oxidise water, it cannot be used for disinfecting water.

Bromine is a heavy dark reddish-brown liquid that upon addition to water forms hypobromous acid (HOBr), the dissociation of the acid resulting in the formation of hypobromite ion (BrO^-). Bromine also reacts with ammonia in water to form monobromamine and dibromamine. No stable tribromamine is formed. Monobromanine is a strong bactericide, almost as strong as free bromine, in contrast to monochloramines. Bromine has been used for the disinfection of swimming pool waters on a limited scale. However, because of its higher cost and less effectiveness, its use for public water supply has not found acceptance.

Iodine is a bluish black solid and its addition to water yields hypoiodous acid (HOI) and hypoiodite (IO). Iodine reacts less with organic matter compared to chlorine and is relatively stable in water. At pH 7, the percentage of iodine, hypoiodous acid, and hypoiodite ion have been reported to be 52, 48, and 0, for a total iodine residual of 0.5 mg/L. Both iodine and hypoiodous acids are equally good disinfectants. Iodine does not react with ammonia to form iodamines, but oxidises ammonia. It also oxidises phenols. Because of these reasons, less iodine is required to obtain free iodine residuals. Iodine has been used for the disinfection of swimming pool waters and small quantities of water in the field. Iodine tablets (e.g., tetraglycine hydroperiodide tablets) have been used by the defence personnel. However, iodine is more costly than chlorine and imparts a typical colour and odour to water, hence not used for disinfection in water treatment plants.

9.16.4 Metal Ions

Several metals including silver, copper, mercury, cobalt, and nickel possess significant bactericidal properties. However, except silver, none has been found suitable for disinfecting drinking water supplies. Silver is relatively ineffective against viruses and cysts in acceptable concentrations. Long detention periods are required but very low concentrations of the order of 15 bg/L of silver ions are sufficient to destroy most organisms. Silver can be introduced into the water either in the form of silver salt or by immersing silver or silver-coated electrodes in the water and applying an electrical potential. Successful applications at 100 V have been reported. As the solubility of most silver salts is adequate, enough silver ions may dissolve which is considered sufficient for most disinfection purposes. However, these disinfection methods are restricted to households or very small water supply schemes (1-10 KLD).

9.16.5 Ozone

Ozone is a faintly blue gas of pungent odour. Being unstable, it breaks down to normal oxygen and nascent oxygen. This nascent oxygen is a powerful oxidising and germicidal agent. Ozone is produced by the corona discharge of high-voltage electricity into dry air. Ozone, being unstable, has to be produced on-site.

Ozone is a disinfectant and is also highly effective in the removal of tastes, odours, colour, iron, and manganese. Ozone acts similarly to chlorine and disinfects by oxidising and destroying the cell wall of microorganisms, resulting in a leak of cellular components outside the cell. This leads to protoplasmic damage of the cell, adversely affecting constituents of the nucleic acids ultimately resulting in depolymerisation. Ozone, being a bioactive oxidising disinfectant, splits into O_2 and O_1 molecules, and O_1 being highly reactive, results in the breakdown of bacterial cell walls by altering the function of carbohydrates and proteins. As ozone reacts with chemical contaminants before attacking the microorganisms, it produces essentially no disinfection residual unless the ozone demand of water has been satisfied. Ozone achieves rapid kills once free ozone residuals are available. Ozone is effective in killing some chlorine-resistant pathogens like *Giardia, Cryptosporidium,* and certain viruses. Ozone does not impart offensive tastes and odours to water, nor does it usually produce toxic substances such as chlorinated hydrocarbons or trihalomethanes (THMs). Further, the efficiency of disinfection by ozone is unaffected by the pH or temperature of the water over a wide range.

Ozone can be added at any of the locations such as pre-oxidation, intermediate oxidation and/or post-ozonation (disinfection) in the water treatment plant. Ozone should be used for pre-oxidation prior to sand filter. Ozonation has many benefits:

i. Removal of iron and manganese
ii. Removal of micro-pollutants, e.g., pesticides, pharmaceutical and personal care products (PPCPs)
iii. Improvement in the performance of coagulation, flocculation, and sedimentation process resulting in lower water turbidity and reduced coagulant dosages
iv. Improved disinfection and reduction in DBPs
v. Elimination of odour and taste causing compounds

Among the disadvantages of ozone treatment are:

(i) Its high cost of production
(ii) Inability to provide residual protection against recontamination
(iii) Its generation is on-site due to instability

However, despite these disadvantages, ozone has been extensively used in developed countries for the disinfection of municipal water supplies as their systems are 24x7 and entry of contaminants is ruled out.

a) CT values for ozonation

Ozone-based disinfection also works in the similar principle that of chlorine disinfection. Hence, C (residual concentration of the disinfectant, mg/L) and T (exposure time, minute) are important factor in designing ozonation system for disinfection in a water treatment plant. There are complex methods to determine CT values in case of disinfection using ozone and few of the methods are as follows (USEPA, 2010):

- T10 Method
- Continuous Stirred Tank Reactor (CSTR) Method
- Extended T10 Method
- Extended CSTR Method

Considering complexity of these methods, following approach can be considered to determine CT values for ozonation.

Cryptosporidium is one of the most resistant pathogens to disinfection, hence it can be safely presumed that if *Cryptosporidium* is removed/killed/inactivated, other pathogens can also be inactivated. CT values for *Cryptosporidium* are presented below for various log removal at various temperatures. Table 9.3 shows CT Values for Inactivation of Cryptosporidium by Ozone.

Table 9.3: CT Values for Inactivation *of Cryptosporidium* by Ozone

S.No.	Log removal	Water temperature			
		15°	20°	25°	≥30°
1	0.5	3.1	2.0	1.2	0.78
2	1.0	6.2	3.9	2.5	1.6
3	1.5	9.3	5.9	3.7	2.4
4	2.0	12	7.8	4.9	3.1
5	2.5	16	9.8	6.2	3.9
6	3.0	19	12	7.4	4.7

Source: USEPA, 2010, Long Term 2 Enhanced Surface Water Treatment Rule Toolbox Guidance Manual

There are limited studies available in India about presence of *Cryptosporidium oocysts* in drinking water. Presence of average count of *Cryptosporidium oocysts* varied from 0.17/100 L in Chennai (2015) to 160/100 L in Amritsar (2019) (Utaaker et al, 2019, Daniels et al, 2015, Anbazhagi et al, 2007). However, maximum count of 1800/100 L *Cryptosporidium oocysts* in Chennai. Considering log credit of 2 (average) for sedimentation, coagulation and flocculation and filtration (WHO, 2022 and USEPA, 2010), additional 0.5-1.0 log removal (credit) due to ozone is recommended. This credit should be used for design purposes for ozone treatment system.

However, there is a need to estimate ozone dose prior to disinfection as ozone will be consumed by organic matter, ammonia, etc. It is recommended that following doses of ozone should be considered:

Optimal concentration to remove organic matter by ozone is at an ozone dose of: O_3/DOC = 1 mg/mg (Source: https://www.lenntech.com/library/ozone/drinking/ozone-applications-drinking-water.htm)

b) Components of ozonation system

There are following four main components of an ozone disinfection system:

i. Oxygen (feed gas) unit
ii. Ozone generator
iii. Ozone contactor
iv. Off-gas ozone destruction unit

However, configuration of these units can vary based on various factors, viz., capacity of water treatment plants, raw water quality, performance of preceding water treatment units, etc.

Ozone in produced by ozone generators normally fed by oxygen concentrators through a high-voltage electric field. This ozone is then monitored in water by an ozone in water analyser. The ozone-enriched gas is directly dosed into the water by using porous diffusers at the bottom of baffled contactor tanks. The contactor tanks, normally about 5 m deep, can provide contact time in the range of 10-20 minutes. Minimum 80% of the applied ozone is possibly dissolved in water, whereas the remainder ozone contained in the off-gas is circulated through an ozone destructor and ultimately vented to the atmosphere (WHO, 2022). Ambient ozone is controlled by ozone sensors; these check if the ozone destructor works sufficiently. Ozone generators are shown in Figure 9.14.

Figure 9.14: Ozone Generator

c) Ozone Injection techniques

Ozone gas can be injected in water in different ways. The two most used techniques are:

I. Venturi
II. Diffuser

A venturi injects the ozone gas in the water via a vacuum. The advantages of a venturi are the compact installation, possible high yield (up to 90%). In the picture below, an example of a venturi system can be found. A side stream injection with pump is used.

A diffuser works under pressure. A diffuser creates a bubble column. The advantages are high yield, simple construction, and advantageous for high flow rates (i.e., drinking water systems). Disadvantages are the required surface area and the need of tall buildings to increase the efficiency.

Ozonation should be followed by chlorination to have free residual chlorine in the range of 0.2-0.5 mg/L while transporting water to households.

d) Slime and biofilm formation

Ozonation processes increases biodegradable part of natural organic matter (NOM) in water as several large organic molecules are broken into smaller biodegradable organic molecules. These smaller biodegradable organic molecules result in increased biodegradable dissolved organic carbon (BDOC) or assailable organic carbon (AOC). Higher BDOC or AOC normally leads to enhanced bacterial growth in water distribution system. It is reported that AOC levels should be less than 100 ppb to prevent/control excessive bacterial growth in water distribution system (LeChevallier et al., 1992) Similarly, by having ozonation unit upstream of filtration, microbial activity and consequently slime formation, particularly in underdrain system, increases in the filter (provided environmental conditions such as pH, dissolved oxygen, and temperature are favourable). Hence, appropriate underdrain systems should be designed so as to prevent clogging while operations of filtration systems.

As ozone adds large quantity of oxygen to the water, a favourable environment for microbial growth is created on the filter media. Hence, it is advisable to locate ozonation unit in water treatment plant (USEPA, 1999). Ozonation units can either be at the end of treatment system if used only for disinfection purposes, or can be located as follows:

i. Pre-ozonation
ii. Intermediate ozonation
iii. Main ozonation

In case ozonation is located at the end of water treatment chain, granulated activated carbon (GAC) should precede it. As explained earlier, this filter will act as biological active filtration (BAC) primarily due to microbial growth on filter media, but have several advantages as explained below:

- Prevention of excessive growth and regrowth of microorganisms (biofilm) in water distribution system
- Removal of NOM that is precursors to disinfection by-product formation due to subsequent chlorination (for having residual chlorine in distribution system)
- Reduction in BDOC/AOC concentration in treated water, thus considerably reducing possibility of regrowth of microorganisms
- Reduction in the demand of residual chlorine of the treated water. This will help in considerably lower levels of disinfection by-product
- Control or minimisation of ozonation by-products
- Removal of algae in treated water

Growth of biomass is high on GAC due to rougher surface characteristics as compared to anthracite, sand, or any other filter medium.

e) Possible layout of water treatment plants with ozonation units

As explained earlier, ozonation will considerably improve treated water quality. However, ozone has substantially low half-life in water and chlorine will be required to take care of microbial contamination in distribution systems. In addition, GAC (also terms as biologically active filtration in this case due to microbial growth on GAC) remains an inevitable unit of treatment involving ozonation. Hence, combination of ozonation, GAC filter, and chlorination should be used wherever ozonation is proposed for optimum treatment. Figure 9.15 shows a possible treatment scheme with ozonation: source water is contaminated with organic matter, ULBs should adopt ozonation as part of water treatment plant.

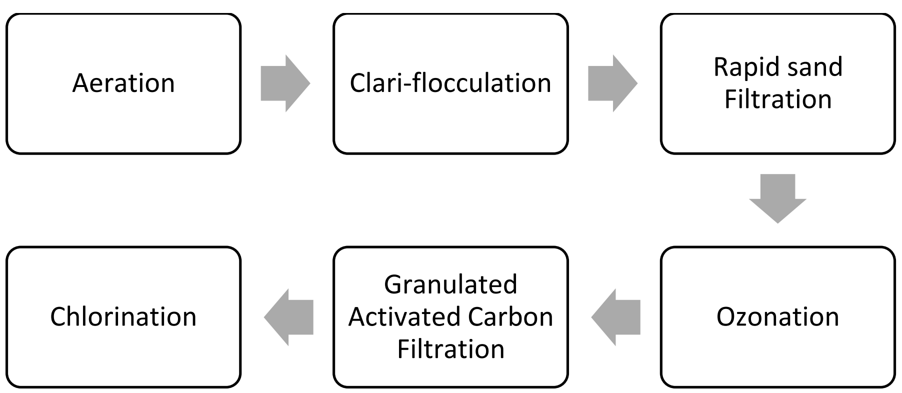


Figure 9.15: Treatment Scheme with Ozonation

Ozonation in TG Halli Water Treatment Plant, Bangalore

The Chama Raja Sagar (CRS) reservoir also known as TG Halli reservoir at TG Halli, is located downstream of the Hesaraghatta Lake near Bangalore. This reservoir has been enlarged from time to time over the years, to partially meet the increasing water supply demand of Bangalore city.

The existing water treatment plant at downstream of TG Halli reservoir is not in operation for a long time as there is no water supply from the reservoir. Presently, TG Halli reservoir also receives contaminated water due to the discharge of untreated/partially treated sewage from the sewage treatment plants at Arkavathi and Kumudavathi, which are located in the catchment area of TG Halli reservoir. In order to revive the drinking water supply from TG Halli reservoir, a '110 MLD Capacity WTP at TG Halli based on Ozone with Granular Activated Carbon Filtration Process' is taken up by demolishing existing units of WTP at TG Halli with major advancements in process technology.

After conventional treatment units such as aeration, coagulation, and flocculation, lamella plate settlers are provided. Clarified water from lamella plate settlers will be brought into a common inlet channel leading to the GAC filters. Dual media filters comprising of GAC and sand are provided to remove residual organics and suspended matter from the water before chlorination and water supply. In TG Halli project, the GAC filter media of 1,400 mm depth is used. The GAC of mesh size 8×30 with surface area of 1,000 to 1,200 m^2/g is used.

In TG Halli project, the ozone generation unit of 15 kg/hour (three units of 5 kg/hour each) is proposed for installation. From this ozone generation unit, pre-ozonation with 0.60 mg/liter and post-ozonation with 5 mg/liter is proposed. The ozone is diffused through the diffusers in post ozonation and through the static mixers in pre ozonation unit. The project is nearing completion and will be commissioned in September 2023.

f) Emerging Technologies: Oxygen Enriched Ultrafine Bubbles /Nanobubbles Technology for Disinfection of Drinking Water and Ozone enriched Microbubbles for Pretreatment of Raw Water

The chlorination is commonly used for disinfection of drinking water, which is producing disinfection by-products, and these are carcinogenic in nature.

Oxygen enriched Ultrafine Bubbles /Nanobubbles or Ozone enriched ultra fine/Nanobubbles (less than 200 nm) deliver higher surface area and remain buoyant in water for days (roughly four times longer than normal diffuser) thus ensuring the residual benefits of maintaining the quality of treated water for a longer period. Therefore, oxygen enriched ultrafine bubbles/nanobubbles have been proven to be alternative disinfectant to chlorination.

In ozone bubble technology, the oxygen formed during decomposition of ozone generates OH radicals that oxidize pollutants. Though ozone ultrafine/nanobubbles are effective for pretreatment as well as disinfection, the amount of residual ozone at the point of usage may arise concerns related to public health and safety as nano bubbles have residual effect. This needs to be further evaluated before application. However, microbubble ozone (10-50 micron) may be considered for pretreatment in lieu of conventional ozonation for pre-treatment and disinfection as microbubble ozone do not have any residual effect as the application of ozone dose is less than the conventional ozone treatment.

The Ultrafine/Nanobubble technology may be considered for disinfection of drinking water as it has residual effect. The micro bubble zone technology may be considered for both pre-treatment and disinfection. The combination of both micro bubble ozone for pre-treatment and ultrafine/nano bubble ozone may also be considered for pre-treatment and disinfection.

However, it is recommended that the Ultrafine/nanobubble technology and micro bubble ozone technologies shall be tested at pilot scale in India under field operating conditions before any large-scale application. Alternatively, the reliable field data for large scale water treatment plants working elsewhere (outside India) may be studied and the technologies validated before large scale application in India.

9.16.6 Ultraviolet Radiation

It was observed that exposure of water to sunlight and artificial light leads to the destruction of organisms. These bactericidal effects of intense sunlight or artificial light are primarily due to ultraviolet (UV) rays. UV light is shorter wavelength (higher energy) than visible. It is called ultraviolet because it is just beyond violet light. Technically, ultraviolet light is defined to be any wavelength of light - also called electromagnetic radiation - shorter than 400 nanometres. UVC (200-280 nm) light inactivates microorganisms by damaging their genetic material (DNA) and rendering them unable to replicate. Ultraviolet radiation may kill a cell, retard its growth, and change its heredity by a gene mutation. The UVA (315-400 nm) and UVB (280-315) are responsible for sun tanning and sun burning.

(a) Dose (Fluence), Irradiance and Contact Time - Theory behind the UV-Tube

To inactivate a given microorganism, the water must be hit with a certain amount or dose of UV light. Dose, more properly referred to as fluence, is determined by variables associated with the design and operation of the UV disinfection device, as well as the characteristics of the water that is treated.

(b) Fluence calculation within the UV-Tube

Fluence is calculated as the product of light intensity at a given wavelength and exposure time. Intensity at any given point is determined by bulb strength and the geometry of the reactor. The greater the thickness of water that the light travels through, the lower the intensity received. The exposure time is governed by the geometry and the hydrodynamics of the reactor. The UV-tube is designed such that the lowest fluence received by any of the water is sufficient to achieve the desired reduction in microorganisms. Fluence may be reported in mW-sec/cm^2, which is equivalent to mJ/cm^2, or in the SI units, J/m^2. Standard doses delivered by a UV drinking water treatment system are between 15 and 50 mW-sec/cm^2.

(c) Intensity

Intensity decreases due to attenuation and dissipation. In other words, the further from the source, the lower the strength of the light, because it is spread over a larger area (due to dissipation) and the lower the strength of the light, because it interacts with molecules in the water (attenuation). While dissipation is predicted simply by the geometry of the UV-Tube, attenuation depends on characteristics of the water. If the water contains a high concentration of materials that absorb UV light, then less UV will be transmitted. The amount of light absorbed per centimetre is expressed as the absorption coefficient (a). As this coefficient increases, transmissivity decreases exponentially, therefore the absorption coefficient of the water plays a very important role in the effectiveness of the UV disinfection device.

(d) Absorption Coefficient

The absorption coefficient describes how much light is lost as it travels through a medium. It can be determined experimentally and is reported in inverse centimetres. The absorption coefficient of pure distilled water is close to zero. NOM, iron, nitrate, and manganese absorb UVC light and will increase the absorption coefficient of a water sample. Absorption coefficients in drinking water would be expected in the range of 0.01 to 0.2 cm^{-1}.

(e) Flow Rate

The higher the flow rate, the shorter the hydraulic detention time, therefore the smaller the dose received by the water. An appropriate flow rate must be determined based on the other characteristics of the water and on the desired dose.

(f) Suspended particles, Particle Associated Microorganisms and Turbidity

Turbidity is a measure of the quantity of particulates in a solution. It is determined by shining an infrared beam of light through a one-centimetre-thick sample and measuring light detected by sensors placed at ninety degrees to the beam. Turbidity is not necessarily correlated with the absorption coefficient. Turbidity is commonly reported in NTU.

Turbidity is often thought to be a limiting feature in ultraviolet disinfection. However, work has shown that particles, as long as they are not UV-absorbers, do not significantly reduce the overall irradiance by either shading or scattering, but only when organisms are embedded within them (Linden, 1998; Emerick, 1999). Particle suspension can increase the apparent absorption coefficient - as measured by a spectrophotometer - by scattering rather than absorbing light (Linden, 1998). This effect can lead to under prediction of design capabilities.

(g) Bulb Types

The maximum UV absorbance of DNA, 260-265 nm, coincides well with peak output of low-pressure mercury arc lamps at 253.7 nm. Two different types of lamps are typically used in water disinfection, medium pressure and low-pressure mercury vapour arc lamp. Table 9.4 displays these differences.

Table 9.4: Comparison between different types of lamps used in water disinfection

Characteristic	Low Pressure/Low Intensity	Medium Pressure/High Intensity
Typical Energy Use:	60 W	5,000 W
Percentage Output at 253.7 nm:	88%	44%
Ozone Production:	NONE	POSSIBLE (quartz can be doped to prevent formation)
Susceptibility to Cooling:	YES	NO
Susceptibility to Cooling:	GOOD	POOR
Benefits:	Efficiency (lower energy requirements)	Smaller, Less Maintenance, Use with Poor Quality Water

Current UV-tube designs use a low-pressure mercury vapour arc lamp.

(h) Bulb Strength

Fluence depends on bulb strength. A 30-watt low-pressure GE T8 bulb emits approximately 5 watts at 254 nm. As the bulb ages its strength will slowly diminish. GE recommends a replacement after 7,500 hours, assuming bulb is turned on for three-hour periods. Each start is expected to decrease the bulbs lifetime, so if the bulb is left on for periods greater than three hours, it may last longer. Conversely, if a bulb is turned on and off for periods shorter than three hours, its lifetime may be reduced to fewer than 7,500 hours. There is a need for more information on the effect of bulb on/off cycles, temperature conditions and general deterioration over time.

Depending upon the dose of radiation and the particular portion of the cell receiving radiation, one or several of the above-mentioned 'three' effects may occur. Wavelength ranging from 2,500 to 2,650 A° is recommended for maximum destruction of cells.

Ultraviolet rays are most commonly produced by a low-pressure mercury lamp constructed of quartz or special glass which is transparent and produces a narrow band of radiation energy at 2,537 A° emitted by the mercury-vapour arc.

Efficient disinfection can be achieved if,

(i) Water is free from suspended and colloidal substances causing turbidity,

(ii) Water does not contain light absorbing substances such as phenols, ABS, and other aromatic compounds,

(iii) Water is flowing in thin films or sheets and is well mixed,

(iv) Adequate intensity and time of exposure of UV-rays are applied.

About 2% of applied energy of ultraviolet rays may be reflected and some energy is absorbed by the impurities present leading to attenuation of radiant energy. Even distilled water will absorb about 8% of the applied energy for a water depth of 30 mm, including a surface reflection of about 2%. The presence of iron even at a low concentration of 1 mg/L may drastically increase absorption by over 80%. A water depth of about 120 mm is recommended for efficient disinfection.

(i) Intensity

The intensity of ultraviolet rays is expressed in terms of germicidal unit which is an intensity of 100 milliwatts (mW) per sq. cm. at wavelength of 2,537 A°. It has been reported that 99.99%, 99%, and 90% kills for Escherichia Coli can be achieved by ultraviolet rays of 3,000, 1,500, and 750 mW per sq. cm respectively. Typically, a 30-watt lamp could achieve 99.9% kill for water flows of approximately 2.5 to 17.0 m3/hr. for water depths ranging from 125 to 880 mm approximately assuming 90% absorption of ultraviolet rays.

(j) Water Depth

For a given-sized UV-tube, determining the best water depth (or weir height) involves a trade-off between residence time and water thickness. The higher the water height, the greater the volume of water in the tube at any given flow rate, and therefore, the greater the average residence time. However, the higher the water height, the greater the attenuation of light and the lower the dose reaching the water at the very bottom of the tube. Since attenuation is proportional to the absorption coefficient, the optimal water height will depend on the absorption coefficient. The optimal weir height (or water depth) is inversely related to the absorption coefficient.

The advantages of ultraviolet radiation are that exposure is for short periods, no foreign matter is actually introduced and no taste and odour is produced. Overexposure does not result in any harmful effects. The disadvantages are that no residual effect is available and there is a lack of a rapid field test for assessing the treatment efficiency. Moreover, the apparatus needed is expensive.

9.17 Disinfection By-Products

Disinfection by-products (DBPs) pose a threat to human and animal health. These are chemical, organic, and inorganic substances that can form during a reaction of a disinfectant with naturally present organic matter in the water. Table 9.5 provide details of DBPs of various disinfectants.

Disinfection processes can result in the formation of both organic and inorganic DBPs. The most well-known of these are the organochlorine by-products, such as trihalomethane (THM) compounds and haloacetic acids (HAAs), related to chlorination. The concentrations of these organochlorine by-products are a function of the nature and concentration of oxidisable organic material in the water, the pH of the water, the water temperature, the free chlorine concentration, its contact time with the organic material.

The types of DBPs that are formed depend on several influential factors listed below.

- The type of disinfectant
- The level of disinfection dose
- The disinfection residue

Effect of reaction time, temperature, and pH is given as follows:

- When the reaction time is shorter, higher concentrations of trihalomethanes (THM), and halogenic acetic acids (HAA) may be formed. When the reaction time is longer, some temporary forms of DBPs may become disinfection end products, such as tribromoacetic acid ($C_2HBr_3O_2$) or bromoform. Haloacetonitriles (HAN) and haloketones (HK) are decomposed.
- When temperatures increase, reactions take place faster, causing a higher chlorine concentration to be required for proper disinfection. This causes more halogenic DBPs to form. An increase in temperature also enhances the decomposition of tribromoacetic acids, HAN, and HK.
- When pH values are high, more hypochlorite ions are formed, causing the effectivity of chlorine disinfection to decrease. At higher pH values, more THM is formed, whereas more HAA is formed when pH values are lower. At high pH values, HAN, and HK are decomposed by hydrolysis, because of an increase in hydrolysis reactions at higher pH values.
- Surface water sources are more susceptible to organochlorine by-product formation than ground water because they receive organic matter in runoff from lake and river catchments. This organic matter comprises mostly humic substances from decaying vegetation, much of which can be in dissolved form as well as in colloid form. The concentration of this organic matter in surface water catchments can vary significantly after rainfall events.

Table 9.5: DBPs of Various Disinfectants

Disinfectant	Organo-halogenic DBPs	Inorganic DBPs	Non-Halogenic DBPs
Chlorine (Cl_2)/ Hypochlorous acid (HOCl)	Trihalomethanes, Halogenic acetic acids, Haloacetonitriles, Chlorine hydrates, Chloropicrin, Chlorophenols, N-chloramines, Halofuranones, Bromohydrins	Chlorate (particularly the application of Hypochlorite)	Aldehydes, Alkanic acids, Benzene, Carboxylic acids
Chlorine dioxide (ClO_2)		Chlorite, Chlorate	unknown
Chloramines (NH_3Cl, etc.)	Haloacetonitriles, Cyano chlorine, Organic chloramines, Chloramino acids, Chlorohydrates, Haloketones,	Nitrite, Nitrate, Chlorate, Hydrazine	Aldehydes, Ketones
Ozone (O_3)	Bromoform, Monobromine acetic acid, Dibromine acetic acid, Dibromine aceton, Cyano bromine	Chlorate, Iodate, Bromate, Hydrogen peroxide, Underbromic	Aldehydes, Ketones, Ketoacid, Carboxylic acids

Disinfectant	Organo-halogenic DBPs	Inorganic DBPs	Non-Halogenic DBPs
		acid, Epoxy, Ozonates	

9.17.1 Total Organic Carbon (TOC) measurement

Drinking water specifications including bacteriological requirements, virological requirements, biological requirements, and pesticide residue limits shall comply with the requirements given in the Bureau of Indian Standards IS 10500: 2012 (Reaffirmed Year: 2018). To monitor this, sampling and analysis of various parameters shall be conducted according to Bureau of Indian Standards BIS 3025 (Part 1 to Part 79) and IS 1622 (1981, reaffirmed 2019). However, parameter like total organic carbon (TOC) is not included in that standard.

Various source waters are 'Rivers, Lakes, Canals, Ground, Rain and Sea, etc.' which are contaminated due to discharge of untreated municipal and industrial water. Consequently, the organic carbon is present in fresh water as constituent of many waste materials and effluents. The organic carbon also arises from living organic matter in fresh water. Total organic compound (TOC) is emerging as an alternate parameter to measure organic load in both raw and treated water and is widely being recognised as an index of organic substance in water.

Furthermore, the raw water containing TOC, when treated in water treatment plant (WTP) with chlorine as disinfectant, DBPs like trihalomethanes (THMs) and haloacetic acids (HAAs) may be formed, which are carcinogenic. The lower the TOC, the better the quality of water and vice versa. Few of the developed countries have notified TOC limits 4-5 mg/L in source water and 2-3 mg/L in drinking water. TOC removal shall be required mainly in WTPs that uses conventional treatment to treat surface water. TOC is used as a surrogate parameter to DBPs therefore a detailed study for the same is needed.

Anna University, Chennai, has been entrusted by the MoHUA (CPHEEO) to carry out a study namely 'Formation, Fate and Treatment methods for DBPs in Water and Wastewater', which will focus on monitoring and removal methods of TOC as well as the application of ozonation as a disinfectant for different concentrations of TOC and other micro-pollutants in raw water sources. The outcome of the study shall be available at the Ministry's website.

9.18 Advantages and limitations of various disinfection methods

Advantages and limitations of various disinfection methods are given in Table 9.6.

Chemical disinfection methods are generally more effective against bacteria and viruses, with little or no effect in the case of chlorination for the inactivation of protozoan pathogens. On the other hand, UV light is very effective against protozoan pathogens with additional effectiveness against bacteria and, to a lesser extent, viruses in water.

9.18.1 Combinations of disinfectants

Combination of disinfectants is known to lead to greater inactivation when the disinfectants are added in series rather than individually. There are also benefits from two or more disinfectants in dealing with a range of different types of pathogens of different sensitivities to disinfectants, e.g., UV is effective for *Cryptosporidium*, but much less effective for many viruses, whereas chlorine is effective for viruses but not so much for *Cryptosporidium*. Further, another benefit from using ozonation and UV treatment in sequence is that ozone can degrade natural organics which cause UV absorption and, hence, allowing the UV dose to be a more effective. A graphical representation of UV and chlorination dosage necessary to inactivate a range of common pathogens is shown in Figure 9.16. It can be seen that there is a benefit in the multi-barrier use of both disinfection methods in the provision of full-spectrum pathogen control.

The Advantages and Limitations of various disinfection methods are given in Table 9.6

Table 9.6: Advantages and Limitations of various disinfection methods

Process	Advantages	Limitations
Chlorination	• Capability and design aspects are well understood.	• Chlorination by-products and taste and odour issues.
	• Established dosing technology.	• Ineffective against *Cryptosporidium*.
Chloramination	• No significant by-product issues.	• Considerably less effective as compared with chlorine.
	• Generally, less taste and odour issues than chlorine.	• Not usually practical as a primary disinfectant.
Ozone	• Strong oxidant and highly effective disinfectant.	• No residual for distribution, possible regrowth in the water distribution system.
	• Additional advantage of destruction of organic micro-pollutants (pesticides, taste, and odour compounds).	• Energy intensive and expensive equipment.
Chlorine dioxide	• Can be more effective than chlorine at higher pH, and less taste and odour and by-product issues.	• Weaker oxidant than ozone or chlorine.
		• Dose limited by consideration of inorganic by-products (chlorate and chlorite).
UV	• Generally, highly effective for protozoa, bacteria, and most viruses and particularly for *Cryptosporidium.*	• Less effective for viruses than chlorine.
	• No significant by-product.	• No residual for distribution, possible re-growth in the water distribution system.

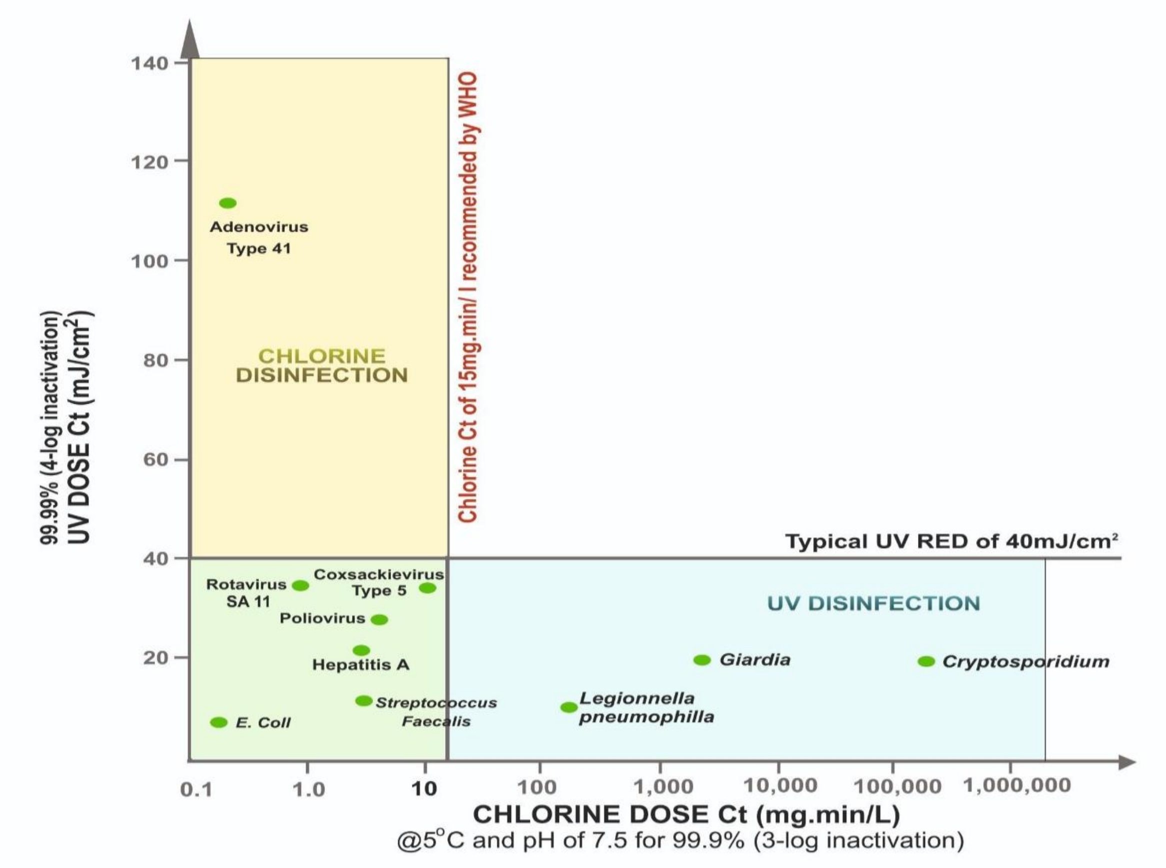


Figure 9.16: Synergistic uses of UV and chlorination disinfection systems

It is to be noted that, while working on combination of disinfectants, care needs to be taken to consider interactions between them, if any. Chlorine is reduced by UV treatment. Although the extent of chlorine reduction is small (e.g., 0.1 to 0.2 mg/L at a dose 40 mJ/cm^2) it is best if chlorine is dosed after UV. Chlorine reacts with ozone to produce chlorate. However, it is unlikely that sufficiently large ozone residual would reach a final chlorination process, for such chlorate formation to become an issue.

UV disinfection can be particularly attractive where there is insufficient space at site for a chlorine disinfection contact tank. Chlorine should not be dosed upstream of a Granular Activated Carbon (GAC) process as the GAC will reduce the chlorine, leaving little or no chlorine residual downstream. Chlorinated water is sometimes used for filter backwashing. There may be some potential for formation of THM with organic material present within the filter. On the other hand, there may be benefits to using chlorinated water to control excessive biological growth in the filter media.

CHAPTER 10: SPECIFIC WATER TREATMENT PROCESSES

10.1 Introduction

Water treatment involves the removal of physical, chemical, and biological contaminants that transform raw water into potable water. The treatment process used in any specific instance depends on the quality and nature of the raw water.

Water treatment processes range from simple processes like sedimentation to complex physico-chemical transformations, as with coagulation. The specific treatment processes include control of algae, taste and odour, removal of colour, softening, removal of iron and manganese, arsenic, hexavalent chromium, mercury, nitrate and fluoride, as well as desalination.

10.2 Control of Algae

10.2.1 General

Algae impart odour, colour, and taste to water. Algae, like *Synura,* cause a perceptible odour; while *Asterionella, Meridion*, and *Tabellaria* produce aromatic odour, and *Dinobryon, Peridinium, Uroglenopsis, Asterionella,* and *Tabellaria* produce fishy odour. Grassy odour is caused by *Aphanizomenon, Anabaena, Gomphosphaeria, Cylindro-spermum*, and *Rivularia*. Septic odour is caused by *Cladophora, Hydrodictyon, Ceratium, Aphanizomenon, Anabaena*, and *Cylindrospermum*. When algae like *Microcystis, Anabaena*, and *Aphanizomenon* die *en masse* and decay, they produce foul odours.

Some algae impart sweet, bitter, or sour taste to water. Algae like *Nitella, Geratium*, and *Synura* give rise to a bitter taste, while algae such as *Chara, Euglena, Aphanizomenon, Microcystis, Cryptomonas*, and *Gomphosphaeria* impart a sweet taste to water.

Algae interfere in the process of flocculation and sedimentation. Algae like *Asterionella* and *Synedra* prevent floc formation. Water containing *Gomphosphaeria* and *Anabaena* needs to be agitated for proper floc formation. They buoy up the flocs and carry them into the filters. They choke the filters and as a result, reduce the filter runs. Algae associated with filter clogging are *Asterionella, Fragilaria, Navicula, Synedra, Cymbella, Diatom, Oscilatoria, Rivularia, Trachelomonas*, and *Closterium*. Algae like *Synedra* and *Oscillatoria* can pass through a rapid sand filter. Algae such as *Euglena, Phacus, Navicula, Nitzschia*, and *Trachelomonas* get through a slow sand filter. These algae in the distribution system cause biological corrosion.

Lyngbya, Anabaena, Cylindrospermum, Nodularia, and *Microcystis* are some of the common toxic algae associated with fish mortality. Hay fever is caused by *Anacystis* and *Lyngbya Contorta*. Gastrointestinal disturbances are also said to be due to algal toxicity.

Algae can be killed by treating the water with suitable chemicals. However, allowing the algae to grow and then adopting algicidal measures has the following disadvantages:

a) Dose of chemical required is greater than that needed if the treatment is adopted at the initial stages of growth;
b) Dead algae decay and produce acute odour problems;
c) Dead algae provide a pabulum for a second crop which is generally more prolific than the first and also more resistant to the action of algicides.

It is, therefore, preferable to take all possible measures to prevent the growth of algae at the initial stage and to reserve the use of algicides as a final treatment.

10.2.2 Causative Factors for Growth

Algal growth is influenced by several factors such as nutrients in the water, eutrophication, the availability of sunlight, the character of the reservoir, and temperature.

10.2.2.1 Nutrients in Water

Nutrients like nitrogen and phosphorus favour the growth of algae. Swamp water or water in contact with decaying vegetation, as well as water polluted by sewage, contains a large number of organic matters favouring certain types of algal growth. Among the various mineral compounds, nitrogen and phosphorus are particularly favourable and are generally brought in by agricultural return waters and some industrial wastes. Algicidal treatments have limited value when the water is rich in such nutrients because the conditions are favourable for the growth of succeeding crops of algae.

10.2.2.2 Eutrophication

Eutrophication is the process whereby lakes become enriched with nutrients that make the water undesirable for human use, both for water supplies and recreation. Limnologists (scientists specifically educated to study surface water sources such as lakes, ponds, etc.) categorise lakes according to their biological productivity. Oligotrophic lakes are nutrient-poor, typical examples are a cold-water mountain lake and a sand-bottomed, spring-fed lake characterised by transparent water, very limited plant growth, and low fish production. A slight increase in plant and algal fertility results in a mesotrophic lake with some aquatic plant growth, greenish water, and moderate production of game fish. Eutrophic lakes are nutrient-rich and produce significant growth in the form of microscopic algae and rooted aquatic weeds that produce a water quality undesirable for body contact and non-body contact recreation.

The process of eutrophication is directly related to the aquatic food chain. Algae use carbon dioxide, inorganic nitrogen, orthophosphate, and trace nutrients for growth and reproduction. These plants serve as food for microscopic animals (zooplankton). Small fishes feed on zooplankton and large fishes consumes small ones. Abundant nutrients destabilise the normal succession and promote blooms of blue-green algae that are not easily utilised as food by zooplankton, thus the water becomes turbid. Floating masses of algae are windblown to the shore where they decompose producing malodours. Decaying algae also settle to the bottom, reducing dissolved oxygen. Even a relatively mild algal bloom can result in the accumulation of substantial decaying scum along the windward shoreline because of the lake's vast surface area. The most damaging aspect of eutrophication is that the process is difficult to slow down once started. Once a lake has become eutrophic, it remains so, for a very long time, even if nutrients from point sources are reduced. Non-point sources are also significant contributors to nutrient loading.

Eutrophic lakes also greatly increase the difficulty of water treatment.

10.2.2.3 Sunlight

Algae require sunlight for their life processes and, hence, the growths are profuse in seasons of intense sunlight. Clear waters favour the growth of algae because they permit the penetration of sunlight to greater depths.

10.2.2.4 Characteristics of Reservoirs

Shallow reservoirs offer more favourable conditions than deep reservoirs because their dissolved nutrients closer to the surface may stimulate algal growth. Irregular margins and shallow areas encourage the growth of aquatic weeds which offer anchorage for the epithetic algae.

10.2.2.5 Temperature Effects

The temperature has a considerable influence on algal growth. The blue-green and the green algae make their presence when the water temperatures reach 20 to 30 °C.

10.2.3 Remedial Measures

10.2.3.1 Preventive Measures

Preventive measures should, therefore, be based on control of those factors such as reduction of the food supply, change of the environment, or exclusion of sunlight though they are not always practicable. Clearwater reservoirs, service reservoirs, and wells can be covered to exclude sunlight, but such a remedy is inapplicable in the case of large reservoirs of raw water. Turbid water prevents light penetration and thereby reduces the algal population, but turbidity provides other organisms protection that also must be removed in the water treatment process. Activated carbon (10.5 to 24.5 kg/ha) reduces the algal population by excluding sunlight but the disappearance of activated carbon in the water may support algal growth again. To a limited extent, the environmental conditions for the growth of algae may be made unfavourable by proper care in the construction and operation of reservoirs.

10.2.3.2 Control Measures-Algicidal Treatment

Algicidal measures may be adopted to control algae in reservoirs. As it has been explained earlier, it is preferable to initiate the treatment in the early stages of algal growth.

(i) Microscopic examination
To decide on the best time at which the water should be treated, it would be necessary to have a regular schedule of microscopic examinations of the water. Such an examination is especially necessary during the season in which algal invasions may be expected.

(ii) Time for treatment
Generally, the practice has been to apply algicides when the total count reaches or exceeds 300 areal (measure of the algal strength/growth rate) units. Areal growth rate is the new biomass per area per time. This is usually used to express growth in a pond or in the ocean. The unit of measure used to quantitatively measure the growth is “areal” unit (mass per area-time, g/m^{2}s). Algae which are known to be particularly troublesome should be eradicated even though the total count is much less than 300 areal units. For example, algicidal treatment is indicated as soon as *Synura*, a type that causes severe smell troubles, is encountered, irrespective of the total count.

(iii) Type of algicides
A large variety of algicides are available and many new algicides are being synthesised. Many of these are complex organic compounds and are credited with specific actions against particular species. Chemicals such as ketones, aldehydes, organic acids, quaternary ammonium compounds, silver nitrate, chlorine dioxide (ClO_2), and rosin amines have also been tried as algicides. However, these are costly and have not come into general use. The most widely used algicides are copper salts, chlorine, and potassium permanganate in small-scale water supplies. The chemical to be used as an algicide should be species selective, non-toxic to aquatic life, particularly fish, harmless to human beings, have no adverse effect on water quality, as well as are inexpensive and easy to apply. The application of copper sulphate and potassium permanganate is discontinued due to several limitations; hence, they should not be used as algicides.

Chlorine
Chlorine is normally a bactericide and is also used as an algicide. Chlorine has a specific toxic effect and causes the death and disintegration of some species of algae. The essential oils

present in the algae are thus liberated and can cause taste and odour problems. Occasionally, these essential oils as well as the organic matter of the dead algae may combine with chlorine to form new or intensified odours and tastes. Such intensification of odours makes the control of algae by chlorine a problem that challenges the ingenuity of the operator. The lethal doses of chlorine for the more common types of algae are given in Table 10.1.

Table 10.1: Amount of Chlorine Required to Destroy Microscopic Algae

Algae	Chlorine dose, mg/L
Aphanizomenon	0.85
Cyclotella	1.00
Melosira	2.00
Dinobryon	0.5
Uroglenopsis	0.5
Synura	0.3

Chlorine may be applied either as a slurry of bleaching powder or as a strong solution of chlorine from a chlorinator and the latter is preferable. Small reservoirs may be treated by applying a slurry of bleaching powder at the influent end or by towing bags containing the bleaching powder in the water. Chlorination for algal growth is more commonly adopted in the pretreatment part of the waterworks. The point of application is generally at the point of entry of raw water into the treatment plant or just ahead of the coagulant feed. Algal growths in raw water conduits can get rid of by heavy doses of chlorine. The addition of chlorine along with coagulant is sometimes practiced.

(iv) Surface aeration of lakes or ponds to control algae growth

Algae blooms are more common during hot, calm, and sunny weather (Figure 10.1). The agitation at the surface that eliminates stagnant spots reduces the spaces accessible for algae to thrive (Figure 10.2). Simply moving the water assists in reducing the number of algae in the pond. Algae prefer quiet, stagnant regions to grow because they dislike moving water or surface agitation. Surface agitation in a pond or body of water is caused by aeration. This is advantageous in several ways. It aids in the elimination of still, stagnant water patches (stratification) and simulates natural breezes.

Surface agitation is also advantageous since it aids in the mixing of algae that are already present in the water column. Algae cannot sit at the water's surface and absorb all of the sunshine it needs for photosynthesis, and it cannot survive without this enormous amount of sunlight. The agitation also helps to de-stratify the pond by mixing up the water and lowering the general pond temperature, making the environment less favourable to algae. Aeration can produce a shift in the carbon dioxide levels within the pond, which can then shift the pH levels, allowing beneficial plants to outcompete the unwanted algae and blue-green algae.

Finally, the agitation serves to refract some of the sunlight that strikes the water's surface. The amount of sunlight that can enter the water column is thus limited. Algae and other aquatic plants struggle as sunlight diminishes throughout the water column. Some will remain, but it will assist in limiting algae and other aquatic plant overcrowding.

Aeration should not be considered as the panacea, but rather as one of many strategies that are often used in conjunction with other ways of algae control. Added aeration is always useful and, most of the time, there will be some additional water quality advantage from added oxygen, including that the increased oxygen levels are beneficial for fish, odour problems, and general pond ecosystem health.

Figure 10.1: Algal Sludge

Figure 10.2: Surface Agitation can be Achieved by Aerators or Recycling Pumps

10.2.3.3 Control of Algae at Water Treatment Plants (WTPs)

Algae in WTPs can be removed by the application of chlorination, ozone, chlorine dioxide, or activated carbon. Pre-chlorination will help in killing the algae and facilitate their settling. Pre-chlorination will prevent the growth of algae in basin walls and will aid in the removal of algae by coagulation and sedimentation because the dead cells of these organisms are more readily coagulated. The chlorine in the settled waters will also destroy slime organisms on the filter sand and thus prolong filter runs and facilitate filter washing. Doses required for this purpose may have to be up to 5.0 mg/L to meet the chlorine demand of water, oxidise free ammonia, etc.; however, the water may have to be de-chlorinated in case of higher residual chlorine, so that it maintains 0.2 to 0.5 mg/L free residual chlorine in the settled water. It is seen that chlorine gas has more potential for oxidation than calcium or sodium hypochlorite.

Most economical results are secured with the use of pre-chlorination for initial disinfection by free residual chlorine and post-chlorination with chlorine dioxide. Chlorine dioxide doses sufficient to give an apparent content of 0.2 to 0.3 mg/L free residual chlorine in the filtered water are adequate. This amount of chlorine dioxide being equivalent in oxidising power to 0.5 to 0.75 mg/L free residual chlorine. Chlorine dioxide, along with chlorine, is effective in pre-oxidation. It reduces the potential for the formation of disinfection by-products (DBPs). The chlorine dose needs to be just enough to immobilise or kill the algae cells. An excessive chlorine dose is likely to rupture the cell structure and bring out intra-cellular compounds and toxins (cell lyses), which further complicates the treatment. The combination of coagulation with aluminium/ferric salts and polyelectrolytes with effective slow mixing and flocculation should ensure the settlement of “intact” dead algae mass from the clarifier. Excessive coagulant dosages during a low turbidity period (raw water) ensure “sweep flocculation” to trap the microscopic algal particles.

Ozone is very effective in the destruction and gives more consistent results since it is a very active oxidising agent. Ozone is slightly soluble in water and hence persists in the treated water for only about 30 minutes.

Micro-strainer

A special process known as micro-straining is being used in some WTPs. The micro-strainer is an open drum. The water is passed through a finely woven fabric of stainless steel. The size of the openings in the mesh determines the size of the plankton to be removed from the water.

Ultrafiltration (UF)

Details of UF are provided in the section related to membrane processes. UF can also be used in combination with granular activated carbon (GAC).

10.3 Monitoring and Control/Removal of TOC in Water

Drinking water specifications including bacteriological requirements, virological requirements, biological requirements, and pesticide residue limits shall comply with the requirements given in the Bureau of Indian Standards, IS 10500:2012 (reaffirmed year: 2018). To monitor this sampling and analysis of various parameters shall be conducted according to BIS IS 3025 (Parts 1 to 79) and IS 1622 (1981, reaffirmed 2019). However, TOC is not included in that standard.

Various sources of water are “rivers, lakes, canals, ground, rain, sea, etc.” which are contaminated due to the discharge of untreated municipal and industrial water. Consequently, organic carbon is present in freshwater as a constituent of many waste materials and effluents. The organic carbon also arises from living organic matter in freshwater. TOC is emerging as the most suitable parameter to measure organic load in both raw and treated water and is widely being recognised as an index of water quality in terms of the total index of organic substance in water.

Furthermore, the raw water containing TOC, when treated in WTP with chlorine as disinfectant, DBP like *Trihalomethanes* (THMs) and haloacetic acids (HAAs) may be formed, which are carcinogenic. Lower the TOC, better the quality of water and vice versa. TOC is the major cause of concern in developed countries which have notified TOC limits 4 to 5 mg/L in source water and 2 to 3 mg/L in drinking water. Therefore, these TOC limits are mainly to control the products formed when disinfection is done with chlorine. TOC removal shall be required mainly in WTPs that uses conventional methods to treat surface water. TOC is used as a surrogate measurement for DBPs, therefore, a detailed study for the same is needed. Nano bubble oxygen or ozonation technology may be considered as an emerging technology for TOC removal.

10.4 Control of Taste and Odour in Water

10.4.1 General

Normally, in summer, the water level depletes below the maximum drawdown level (MDDL) in the reservoir. In such situations, the utility even consumes the water from the dead storage of the reservoir and, therefore, the colour, taste, and odour problem prevails. Colour, taste, and odour are formed due to decay of inundated vegetation at the bottom of the dam/impounding reservoir. The problems of taste and odour (one co-exists with the other) are more intensive and more frequent in surface water sources. Taste and odour are caused by dissolved gases like hydrogen sulphide, mercaptans, methane, organic matter derived from certain dead or living microorganisms (blue and green algae), decomposing organic matter, industrial liquid wastes containing phenols, cresols, ammonia, agricultural chemicals, high residual chlorine, and chloro-phenols. It is possible that some of the dissolved gases might be found in groundwater also.

Biological organisms are one of the most common causes of taste and odour in water. *Diatomaceae,* with *Asterionella* and *Synedra, Actinomycetes*, and free-swimming nematodes are the principal offenders causing earthy or musty odour. Apart from algae, decomposing leaves, weeds, or grasses also cause odour. Vegetation that grows in the low-water areas in the reservoir subsequently gets submerged and decomposes resulting in odour. Chemical and refinery effluents have the greatest potential for odour, followed by domestic sewage. Odour tests indicate that only a small quantity (in mg/L) of these materials is needed to produce a detectable odour.

In short, taste and odour producing materials in water are chemical compounds of many varieties with different physical and chemical characteristics present in water because of direct pollution or biological activity. Most of these compounds are in solution and some exist in the form of particulate and colloidal compounds. Those in solution are comparatively more difficult to remove.

10.4.2 Control of Taste and Odour

Preventive and corrective treatment of raw and processed water is necessary for the control and elimination of taste and odour problems. Wherever possible, preventive steps like control of microorganisms are to be undertaken where the source of raw water is from rivers, reservoirs, or lakes and there is a control on effluent discharges. Special treatment is given to water in the treatment plant for the removal of odours by aeration, oxidation by chemicals, or adsorption by activated carbon.

10.4.3 Corrective Measures

Odours can be removed by mechanical aeration, oxidation by chemicals like chlorine or its compounds, ozone, permanganate, and adsorption of odour by agents such as activated carbon, floc, and clays. For removing dissolved gases like hydrogen sulphide and volatile matter, aeration can be practiced at the start of water treatment. Free available residual chlorination at the pre-chlorination or post-chlorination stage can bring about complete elimination of taste and odour. Inadequate chlorination will only intensify the odour of water containing phenolic compounds, tannin, and lignin. Even with breakpoint chlorination, it may not be possible to remove taste and odour from water in certain cases. Such compounds can be removed by super-chlorination. Super-chlorination is normally done either at the reservoir outlet or the WTP inlet to bring the maximum chlorine concentration and the maximum contact time together to effect oxidation. This should be followed by de-chlorination using sulphur dioxide or sodium sulphate to reduce the residual chlorine to acceptable limits.

Chlorine dioxide has been found extensively efficient and the general dosage values range from 0.2 to 2.0 mg/L. Chlorine dioxide gas is released in water on site by the inter-action of a solution of sodium chlorite ($\mathrm{NaClO_2}$) with a strong chlorine solution of 6000 to 7500 mg/L.

$$2NaClO_2 + Cl_2 \rightarrow 2ClO_2 + 2NaCl \tag{10.1}$$

Though the theoretical ratio of chlorine to sodium chlorite is 1:2.6, values between 1:2 and 1:1 are employed in practice. It is applied at the first stages of the treatment plant. Thereafter, the final desirable residual chlorine may be adjusted by simple chlorination after filtration. Ozone at dosages of 1.0 mg/L has also produced good results. Chlorination is useful in the removal of phenol tastes.

10.4.4 GAC

The preferred method of treatment for taste and odour removal is activated carbon (preferably GAC). Odour-producing substances that cannot be removed by oxidation are physically adsorbed onto the surface. This treatment is usually applied before filtration. The contact time varies from 10 to 60 minutes. Activated carbon performs well at lower pH values. A bed of carbon or suspension kept in circulation could be used. The active surface must be preserved from the coating by other chemicals. Application of carbon can be made before sedimentation if the taste and odour are severe and frequent and in certain cases after sedimentation. The approximate dosage for routine continuous application as the suspension is 2 to 8 mg/L; for emergency treatment, 20 to 100 mg/L. Carbon beds are generally 1.5 to 3 m deep with sizes of 0.2 to 0.4 mm with loadings of about 4.8 $m^3/hr/m^2$ of bed. Filtration rates range from 7.2 to 15 $m^3/hr/m^2$ with expected efficiencies of about 90%. As many variables are involved, pilot plant tests are necessary. Carbon can also be used as a polishing agent to remove residual odours after other treatments.

Variables such as pH, temperature, quantity, and type of organic matter in the influent water and detention time have a marked effect on the efficiency of removal of odorous materials.

10.5 Removal of Colour

Colour in water may be due to natural causes or as a result of human activity. Waters occurring in peaty soils acquire colour because of the presence of colloidal organic matter. Colour is also due to mineral matter in solutions, as a colloid, or in suspensions as in the case of groundwater in certain areas. Waters containing oxidised iron and manganese impart characteristic reddish or black colour. Heavy growths of algae may also impart colour to the water. Discharge of industrial wastes or heavy sewage pollution can also bring in colour. The constituents in coloured water can consist of natural organic matter (NOM) or synthetic organic matter (SOM). NOM mostly consists of fumic and fulvic acids having a low molecular weight.

Colour due to iron and manganese may be removed by specific treatment for the removal of these constituents as discussed in Section 10.6. Water that is coloured because of the growth of algae has to be treated to eliminate the source by control of the algae as discussed in Section 10.2.

Coagulation at a low pH range by chemicals such as alum/PAC or ferric salts and polyelectrolytes (sweep flocculation) is used for removing colour due to colloidal organic matter (NOM). Ferric coagulants are generally superior to alum. After the removal of the colour colloids, the pH of the water will have to be corrected by treatment with lime. The colour colloids are often stabilised at a high pH value and, hence, the addition of lime to aid coagulation is fraught with danger in the case of waters that are coloured. It is essential that laboratory tests should be conducted to determine the most suitable chemical and its optimum dosage in the given conditions.

Treatment with GAC, as described earlier, is effective against most problems of colour in water. Carbon removes the colouring matter by adsorption. Application has been discussed in Section 10.4.4.

10.6 Softening

The hardness of water is due to the presence of calcium and magnesium ions in most cases. Bicarbonates, sulphates, and chlorides are the anions associated with the hardness. Calcium and magnesium associated with bicarbonates are responsible for carbonate hardness and that with sulphates, chlorides, and nitrates contribute to non-carbonate hardness. The purpose of softening is to remove these salts from the hard water. The acceptable limit and permissible limit (in the absence of any alternate source) of total hardness are 200 and 600 mg/L, respectively, as per drinking water quality standards (IS 10500:2012).

The two methods ordinarily used are lime and lime-soda softening and ion exchange softening.

10.6.1 Lime and Lime-Soda Softening

This process is seldom employed in India; however, wherever water characteristics and other limitations exist then this can be used with precautions. More often, it is used for industrial use where the water has to be fed to the boiler since it also reduces colloidal silica. Softening with these chemicals is used particularly for water with high initial hardness (greater than 600 mg/L) and suitable for waters containing turbidity, colour, and iron salts because these have a tendency to inactivate the ion exchange bed, by a coating on the granules. Lime-soda softening cannot, however, reduce the hardness to values less than 40 mg/L while ion exchange softening can produce zero-hardness water. Computation of chemical doses is enclosed in **Annexure 10.1**.

10.6.2 Ion Exchange Softening

The ion exchange process is the reversible interchange of ions between an exchange medium and a solution and this process is used extensively in water softening. The hardness-producing ions preferentially replace the cations in the exchangers and hence this process is also known as "base exchange softening". The ion exchange can produce water of zero hardness. There is only a temporary change in the structure of the exchange material which can be restored by regeneration. The ion exchanger can work on the hydrogen or sodium cycle, the hydrogen ions being released into the water in the former case and the sodium ions in the latter. The regenerant agents are acid and sodium chloride, respectively. In general, the ion exchange materials used in softening, also called zeolites, are hydrated silicates of sodium and aluminium having the formula: $xNa_2O.yAl_2O_3.zSiO_2$. The reaction can be depicted as follows:

$$Na_2Z + \frac{Ca^{++}}{Mg^{++}} + 2A^- \leftrightarrow \frac{Ca}{MgZ} + 2Na^+ + 2A^- \quad (10.8)$$

Where A^- represents the relevant anions of bicarbonates, sulphates, or chlorides and Z represents the anionic part of the zeolite.

Treatment with zeolite thus increases the dissolved solids in the ratio of 46:40 of the hardness removed. The reverse equation operates during the regeneration resulting in a strong solution of calcium and magnesium salts, which is run to waste.

a) Inorganic zeolites

The two common inorganic zeolites are the natural and synthetic types. The natural zeolite is available as green sand while the synthetic or gel type is obtained by the reaction of either sodium or aluminium sulphate with sodium silicate, which after drying is graded to suitable sizes by screening. A cubic metre of green sand weighs 1600 kg with a specific gravity of 2.1 to 2.4 and for regeneration, it requires 3.5 to 7 kg of salt for every kilogram of hardness removed. The synthetic inorganic zeolite weighs 900 to 1100 kg/m^3. The relevant exchange capacities and regenerant requirements are given in Section 10.6.2. (d).

b) Organic zeolites

They are lighter than the inorganic zeolites weighing 500 to 800 kg/m3. These consist of sulphonated-carbonaceous material and sulphonated styrene-type resins which have excellent cation-exchange properties, requiring for regenerating 2 to 4 kg of salt for every kilogram of hardness removed. These are resistant to attack by acid solutions and hence can be regenerated with acid also. They can be used for waters with a wide pH range. The loss due to attrition is negligible compared to the synthetic inorganic zeolites.

c) Raw water characteristics

For application to ion exchangers, the raw water should be relatively free from turbidity (≤1 NTU), as otherwise, the exchange material gets a coating that affects the exchange capacity of the bed. The desirability of using filters before zeolite beds or resorting to more frequent regeneration would depend upon the level of turbidity. Metal ions, like iron and manganese, if present are likely to be oxidised and can coat the zeolites, thus deteriorating the exchange capacity steadily since the regenerant cannot remove these coatings. Oxidising chemicals like chlorine and carbon dioxide, as well as low pH in the water, tend to adversely affect the exchange material, particularly the inorganic types, and the effect is more pronounced on the synthetic inorganic zeolites.

d) Design criteria

The design criteria for a softening system are based upon the following:

(a) The required flow rate:
(b) The influent water quality;
(c) Desired effluent water quality;
(d) Exchange capacity and hydraulic characteristics of the exchanger;
(e) Period between regenerations;
(f) Type of operation;
(g) Number of units required;
(h) Rate, time of contact, uniformity, and concentration of brine application;
(i) Rate and volume of rinse; and
(j) Quality of regenerant.

A softening unit is similar to a rapid sand filter unit regarding hydraulics and equipment.

The volume of exchange material (*E*) to be used in cubic metre is calculated by the formula:

$$E = \frac{QH}{1000G} \tag{10.9}$$

Where,
Q = volume of water to be treated between regeneration, m^3
H = hardness of water, mg/L
G = exchange capacity of the material, kg/m^3

Generally, ion exchange beds are encased in shells, shell diameter, and bed depth being adjusted to maintain a rinse rate of flow in the range of 0.15 to 0.30 m/min. The vertical units are 2 to 3 m in diameter while the horizontal ones are 3 m in diameter and 8 to 9 m long. The ion exchange bed has a depth of 0.6 m usually and is placed over supporting gravel (size depending upon the composition of the exchange material but with similar specifications as those for rapid gravity sand filters) of 0.30 to 0.45 m depth with an underdrain system at the bottom for collecting softened water. After the softening cycle, the softener should be backwashed for 3 to 5 minutes to loosen the exchange resin and remove particulate matter. The rate of backwash should ensure at least 50% bed expansion.

In the ion exchange softening process, which uses regeneration with sodium-based compounds, about 0.5 mg/L sodium is added to water for each milligram per litre of hardness. Hence, after treatment, water that has 400 mg/L of total hardness will have about 200 mg/L of sodium. It is recommended that the hardness of treated water should be slightly less than 200 mg/L which will also comply with IS 1050:2012. Nevertheless, it also means that for every litre of water intake, there would be 100 mg of sodium intake (to bring down total hardness from 400 to 200 mg/L). Persons having restricted sodium diets because of health concerns may not be able to tolerate high sodium water. Therefore, drinking and cooking with soft water is normally avoided and, on the contrary, hard water is provided for drinking and cooking purposes. It is also not recommended to repeatedly use softened water for plants, lawns, or gardens due to the sodium content.

This restricts the use of soft water, hence, ion exchange for softening purposes should be used judiciously and avoided in normal cases.

10.6.3 Combination of Lime and Zeolite Softening

For waters that contain a large carbonate hardness, a combination of lime and zeolite softening can be practiced. The lime treatment, which is applied first, removes by precipitation a large part of the carbonate hardness, simultaneously decreasing the amount of dissolved solids in the water. After leaving the lime reaction tank, the water is settled and filtered and then passed through the zeolite softeners which by base exchange remove the residual carbonate hardness and all the non-carbonate hardness.

The combination of lime and zeolite offers the following advantages:

a) It gives water a lower hardness than can be obtained by lime and soda ash treatment.
b) It reduces the amount of total dissolved solids (TDS) which the zeolite treatment alone would not do.
c) It gives a lower cost of chemicals than with lime and soda ash and possibly lower than with zeolite alone, depending on the relative costs of salt and lime.

Surveys carried out in other countries have brought out the fact that the benefits of savings of soap alone justify the expenses of softening on a municipal scale. Other benefits like good public relations add to the attractiveness of the proposition. The practice has not, however, caught up with this trend even in those countries. With greater demands for higher quality water, water softening may have to be carried out on a municipal scale also.

10.7 Removal of Iron and Manganese

Appreciable amounts of iron and manganese in water impart a bitter characteristic and metallic taste, and the oxidised precipitates can cause the colouration of water which may be yellowish brown to black and renders the water objectionable or unsuitable for domestic and many industrial processes. In addition, staining of plumbing fixtures and laundered materials can also result. The carrying capacity of pipelines in the distribution system is reduced due to the deposition of iron oxide and bacterial slimes as a result of the growth of microorganisms (iron bacteria) in iron-bearing water. The acceptable limits of iron and manganese are 1.0 and 0.1 mg/L, respectively, as per IS 10500:2012. Iron is present mostly in groundwaters and its concentration is shown in Figure 10.3.

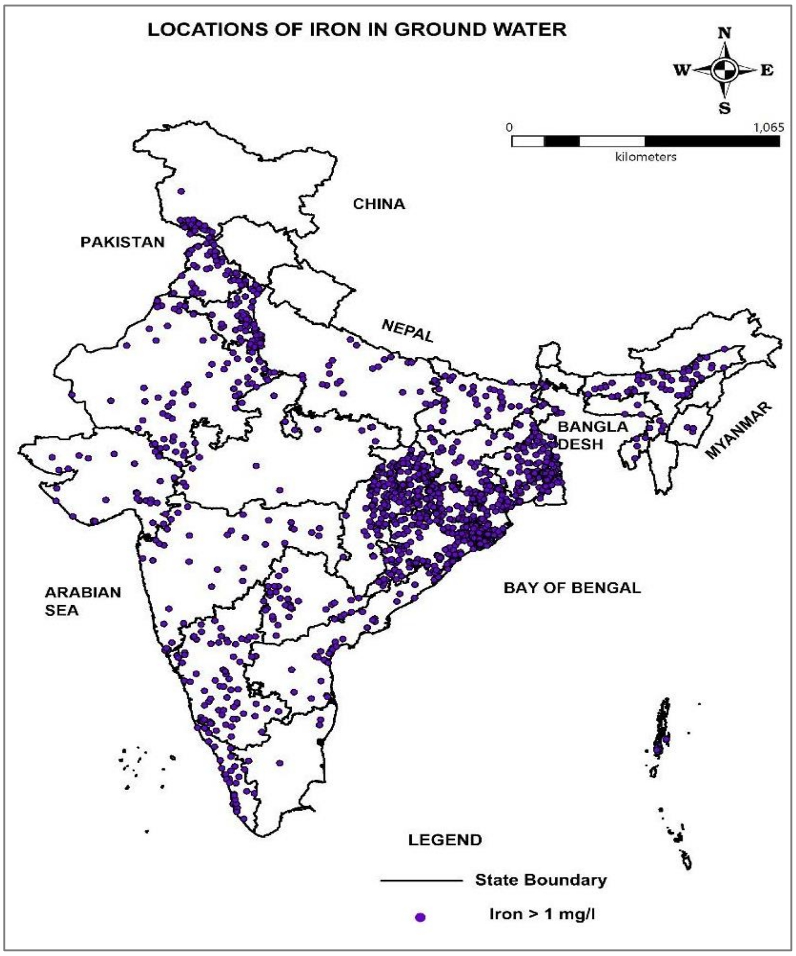


Figure 10.3: Location of Iron in Groundwater
(**Source:** *CGWB.*)

10.7.1 Sources and Nature

Iron and manganese occur in certain underground waters and springs, either alone or in association with organic matter, iron is generally predominant when they are together. They can also be found in surface waters occasionally. Iron and manganese are found in solution in water derived near the bottom of deep lakes, where reducing conditions develop. These conditions are usually seasonal. The presence of iron can also result from the discharge of certain industrial waste or mine drainage. Iron and manganese in groundwater are attributed to the solution of rocks and minerals, chiefly oxides, sulphides, carbonates, and silicates of these metals. This dissolution is enhanced by the presence of dissolved carbon dioxide present in groundwater.

Iron exists in water in two levels of oxidation: (i) as the bivalent, ferrous iron (Fe^{++}) and (ii) as the trivalent, ferric iron (Fe^{+++}), the latter generally occurring in precipitated form. Therefore, in clear groundwaters, if iron is present, it is all ferrous iron. Manganese is also naturally found in water in two oxidation states, bivalent and quadrivalent, with the latter being very sparingly soluble. Iron forms complexes of hydroxides and other inorganic complexes in solution with substantial amounts of bicarbonate, sulphate, phosphate, cyanide, or halides. The presence of organic substances induces the formation of organic complexes and chelates, which increase the solubility of iron and manganese.

The terminology of organic iron and manganese is used when difficulties in oxidation are encountered. There are no analytical techniques for the determination of organic iron or manganese.

Waters with high alkalinity have lower iron and manganese contents than waters with low alkalinity. If the water contains significant amounts of hydrogen sulphide, little or no iron or manganese is found in the solution, as most of it is precipitated.

10.7.2 Removal Methods

Chemical analysis of water alone may not always provide a clue to the removal method to be adopted. Hence, it is advisable that laboratory and pilot plant studies are made before any particular method is used. Oxidation by aeration or the use of chemicals like chlorine, chlorine dioxide, or potassium permanganate followed by filtration alone or by settling and filtration can bring about the precipitation of iron and manganese and their removal. The use of zeolites, as well as catalytic oxidation, also serves the purpose.

10.7.2.1 Precipitation by Oxidation

Iron or manganese in water in the reduced form is converted to soluble ferric and manganic compounds by oxidation, and these are removed by filtration alone or by sedimentation and filtration. The reaction period is about 5 minutes or less at a pH of 7 to 7.5, and 0.14 mg of oxygen is needed to convert 1 mg of ferrous iron to ferric hydroxide. The rate of oxidation of ferrous iron by aeration is slow under conditions of low pH and increases 100% for every unit rise of pH. Rates of precipitation and flocculation are accelerated in practice by contact and catalysis. The contact beds for deferrization are normally 2 to 3 m deep, operating at a surface loading of 40 to 70 $m^3/d/m^2$ with the contact medium sizes from 50 to 150 mm. Accumulation of iron and manganese is flushed out by rapid drainage after filling the bed to near the overflow level. Sedimentation before filtration will be necessary when the iron content exceeds 10 mg/L. A settling period of two to three hours is adequate. The water has to pass through filters (gravity or pressure type) with a 75 cm depth of sand or sand and anthracite. Filter rates are usually 6 to 9 $m^3/h/m^2$.

Oxidation of iron can be inhibited possibly due to the binding of ferrous iron by organic substances and ammonia which behave like tannic, gallic, or ascorbic acids. All the organic material has to be oxidised before any perceptible oxidation of iron can be affected. Chlorination of many iron-bearing waters can bring about the oxidation of organic matter and other reducing agents facilitating the oxidation of ferrous iron. Deeper filter beds up to 2 to 2.5 m with a sand size of 0.6 mm have also been used with good results. In many waters, especially containing organics, pre-chlorination ahead of coagulation, sedimentation, and filtration at pH values between 6.7 and 8.4 usually will ensure iron removal to acceptable limits.

The plants will require washing of filter medium. The necessity of washing is ascertained as and when there is overflow through the overflow pipe provided in the filter compartment of the units. The interval between successive washings varies and depends on the initial turbidity and iron content. Experience indicates a closer interval of one week for high turbidity with an iron content of around 40 mg/L and 1 to 2 months for waters with low turbidities with an iron content of less than 10 mg/L. Washing of filter medium involves the removal of top 5 to 10 cm filter medium and washing it manually with water to free it from sediment and replace the same in position. The filter media needs washing/ replacement once in 6 to 24 months depending on the iron content in raw water.

Manganese removal requires a pH adjustment up to 10.4 to 10.6, and 0.29 mg of oxygen is needed to convert 1 mg of manganese. Pre-chlorination to free residual values up to 0.7 to 1.0 mg/L will affect the oxidation and precipitation of manganese.

10.7.2.2 Zeolite

The method is applicable if the iron is present in a reduced state and in a soluble form in raw water. Such waters are encountered from the bottom strata of deep reservoirs or ground waters. It is usual to limit the application of this process to water having not more than 1 mg of iron or manganese for every 30 mg of hardness up to a maximum of 10 mg/L of iron or manganese.

The process consists of the percolation of the water through a bed of zeolite, which takes up the iron and manganese by a process of ion exchange. The base exchanger can be of the siliceous, carbonaceous, or synthetic resin type. Air should be excluded from the system to prevent the deposition of colloidal oxides on the ion exchange material. Therefore, airlifts, open tanks, or pneumatic tanks should not be used preceding the ion exchanger. If the water is to be softened, then the zeolite process offers a very simple method of iron and manganese removal as it can be carried out under pressure; therefore, usually obviates the necessity for double pumping, as required in most other processes. Many zeolite plants have been installed principally for iron and manganese removal, with softening being of secondary importance. The removal of iron and manganese is almost complete, and the exhausted bed of ion exchange material being regenerated with a salt solution.

10.7.2.3 Catalytic Method

This method has limited application, but is of value if the content of iron and manganese is low, and if it is desirable to treat the water under pressure. It is applicable in the case of clear deep well waters, where the iron is held in solution by carbon dioxide. In municipal use, it is usual practice to restrict the use of this method to waters whose content of iron or manganese is not greater than 1 mg/L. For household use or smaller plants, it may be used with waters containing up to 10 mg/L of iron or manganese. The removal of iron and manganese is accomplished without affecting the hardness of the water, as this process is entirely one of oxidation and filtration and does not involve base exchange. The method consists of percolating the water through suitable contact materials which oxidise the iron and manganese. These contact materials, which are sold under various names, are made by treating a siliceous base exchange material successively with solutions of manganese chloride and potassium permanganate. They may be housed in a separate filter or a layer of this material may be sandwiched in the sand bed of a pressure filter. By percolation through this bed, iron and manganese are oxidised and also filtered out. At intervals, the filter has to be backwashed to remove the deposits. The backwash rates are generally of the order of 21 $m^3/h/m^2$. When the bed loses its capacity for oxidation of iron and manganese, it can be regenerated by treatment with a potassium permanganate solution.

Iron can also be removed at high flow rates, using nanostructured materials.

10.7.3 Iron Removal Plants

When the question of iron removal is under consideration for community water supply, it is important to decide and cover what other treatment of the water, if any, is necessary or desirable. Considerable free carbon dioxide and toxic substances are usually present in ferruginous waters. Hence, it is not advisable to remove iron alone, as leaving the free carbon dioxide which can cause corrosion of mains and pipes. The means by which iron, free carbon dioxide, and other toxic substances are removed from water in community systems consist substantially of their oxidation and removal of free carbon dioxide, followed by precipitation and its separation by sedimentation and/or filtration. Aeration may suffice for the preliminary precipitation but may not be adequate when concentrations are high and pH correction may be required by the addition of lime. The community water supply scheme makes provisions to meet such requirements and comprises a raw water storage tank, cascade tray aerators, chemical doses, sedimentation basin, filtration, and disinfection. Design of iron removal unit is enclosed in **Annexure 10.2**.

Tray aerators are commonly used for aerating water. The trays are designed for an aeration rate of 1.26 $m^3/m^2/h$ and spaced at intervals of 1 m. Then the water is settled in a sedimentation basin having a detention period of 2.5 hours. The clarified water is filtered through a rapid sand filter having sand of effective size of 0.6 to 0.8 mm and uniformity coefficient 1.3 with an effective depth of 1.2 m. The head of water above the sand is 1.35 m and the rate of filtration is 5 $m^3/m^2/h$. The minimum backwash rate is 35 $m^3/m^2/h$ and the total head required for filter wash is 12 m.

The sand is supported over a gravel layer of depth 0.39 to 0.62 m, and it is arranged as per Table 10.2:

Table 10.2: Arrangement of Sand

Size	Depth
65-38 mm	13-20 cm
38-20 mm	8-13 cm
20-12 mm	8-13 cm
12-5 mm	5-8 cm
5-2 mm	5-8 cm

10.7.3.1 Package Iron Removal Plants

Many package iron removal plants have been installed by authorities in various parts of the country. The plants are predesigned for specific requirements and are successfully being operated and maintained. There are various technologies recommended in the "*Handbook on Drinking Water Treatment Technologies*" published by the Department of Drinking Water and Sanitation, such as treatment units developed by NEERI, CMERI, CSIR-NEERI, IMMT, DRDO, CPERA, etc.

NEERI has designed package iron removal plants having different capacities of 0.5 m^3/h, 1.0 m^3/h, 1.5 m^3/h, and 2.0 m^3/h depending upon the requirement of treated water. The plants are designed in rectangular/circular shapes having an aeration chamber, collection chamber, settling chamber, and filter. The settling chamber is provided with a plate settling device to enhance settling and reduce the detention time thereby reducing the dimension of the settling chamber. The aeration chamber contains media of size 2.0 to 5.0 cm gravel/stone chips to increase the surface area of the air–water interface. The iron contaminated water trickles over aeration media through a spraying device. The aerated water flows through pores over the baffle plate to the collection chamber to the settling chamber to filter. The filter bed of 20 cm depth contains sand media of size 0.8 to 1.4 mm, supported by a 5 cm deep gravel bed of size 0.8 to 1.0 cm. The filter is cleaned by making a backwash connection with a hand pump, scraping the sand bed manually, and opening the sludge scouring valve.

10.8 De-fluoridation of Water

Fluoride is one of the few substances that has been proved to have a substantial impact on human health when consumed through drinking water. Fluoride has good effects on teeth at low concentrations in drinking water, but excessive fluoride exposure in drinking water, or in combination with fluoride exposure from other sources, can have a number of negative health effects.

Excessive fluorides in drinking water may cause mottling of teeth or dental fluorosis, a condition resulting in the discolouration of enamel, with chipping of the teeth in severe cases, particularly in children. In Indian conditions, where the temperatures are high, the occurrence and severity of mottling increase when the fluoride levels exceed 1.0 mg/L. With higher levels, skeletal or bone fluorosis with its crippling effects is observed. The chief sources of fluorides in nature are (i) fluorapatite (phosphate rock), (ii) fluorspar, (iii) cryolite, and (iv) igneous rocks containing fluorosilicates. Fluorides are present mostly in groundwaters and high concentrations have been found in parts of Andhra Pradesh, Bihar, Gujarat, Haryana, Karnataka, Kerala, Madhya Pradesh, Maharashtra, Punjab, Rajasthan, Assam, and Tamil Nadu in the country (Figure 10.4). While the majority of values for fluoride concentrations in fluoride-affected areas range from 1.5 to 6 mg/L, some values are as high as 16 to 18 mg/L, and in one solitary instance, even 36 mg/L has been reported. The acceptable limit and permissible limit of fluoride in drinking water, as per IS 10500:2012, are 1.0 and 1.5 mg/L, respectively.

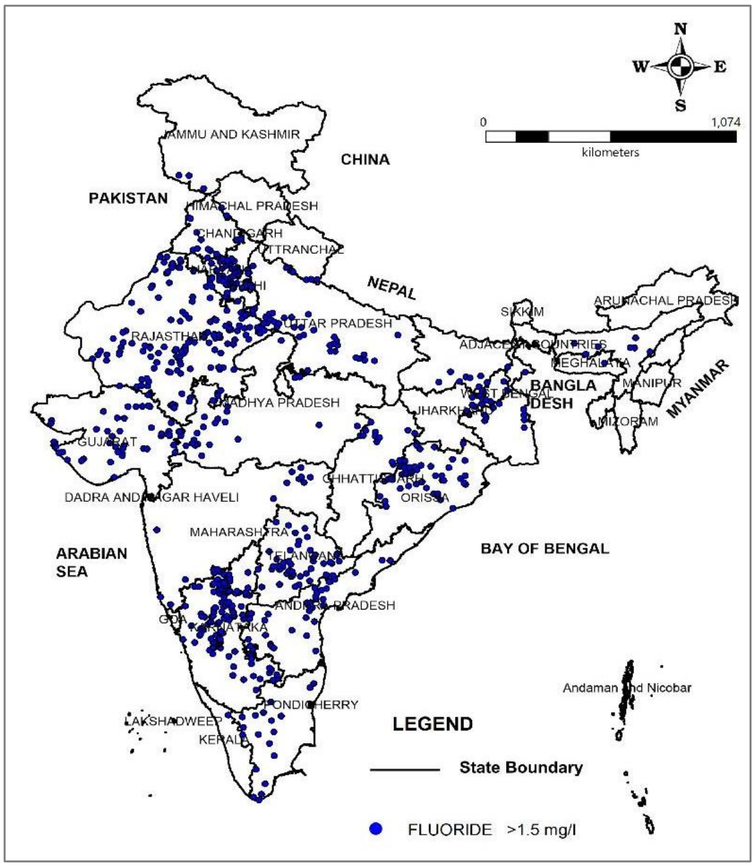


Figure 10.4: Location of Fluoride in Groundwater
(**Source:** *CGWB.*)

10.8.1 Removal Methods

The removal of excessive fluorides from public water supplies or individual water supplies is necessary on public health grounds. For fluoride-free water in the community, the first option must be the provision of a safe source having acceptable fluoride levels, transporting water from safe sources, blending high-fluoride with low-fluoride water to reduce fluoride level below 1 mg/L, and rainwater harvesting. After the exhaustion of these options, water treatment should be appropriately planned and implemented. This is a problem, particularly in rural/peri-urban areas and hence the simplicity of operation, cost-effectiveness, and applicability to small water supplies are of main concerns.

There are several de-fluoridation processes available and in practice globally. Several treatment techniques, such as adsorption, coagulation and precipitation, ion exchange, and membrane filtration, have been developed and implemented. However, the sustainability of these technologies is a major concern due to factors such as a lack of technical support, inadequate operation and maintenance, and improper monitoring.

The common methods used for the removal of fluoride from drinking water are categorized as follows:

- Adsorption and ion exchange
- Coagulation and precipitation
- Membrane filtration processes
- Capacitive De-Ionisation (CDI)

Handbook on Drinking Water Treatment Technologies (2023) published by the Department of Drinking Water and Sanitation, Ministry of Jal Shakti, Government of India, should be referred for de-fluoridation technologies.

10.9 Removal of Arsenic

Arsenic is a metalloid that occurs naturally and is very mobile in the environment. Its mobility is heavily influenced by the type of mineral present in the environment, oxidation state, and mobilisation mechanisms. Arsenic exists in four oxidation states: arsenite (As^{3+}), arsenate (As^{5+}), arsenic (As^{0}), and arsenide (As^{-3}). The predominant arsenic species found in water are inorganic arsenite and arsenate.

Arsenic exposure in humans can occur through ingestion of contaminated food and water, inhalation, and absorption through the skin. If consumed in greater amounts than the permissible limits, arsenic can cause various health disorders in humans such as respiratory distress, cardiac problems, gastrointestinal effects, anemia and leucopenia, skin disorders leading to hyperkeratosis (warts or corns on the palms and soles), and areas of hyperpigmentation interspersed with small areas of hypopigmentation in the face, neck, and back. Chronic exposure to arsenic can result in neural injury, skin cancer, and lung cancer.

According to the Central Ground Water Board, arsenic levels greater than 10 µg/L (ppb) have been detected in groundwater samples from 153 districts in 21 Indian states/UTs (Figure 10.5). However, the middle, lower, and deltaic regions of the Ganga basin are the most affected. Arsenic is also found in several other regions in the recent past whose source has been related to phosphate fertilisers.

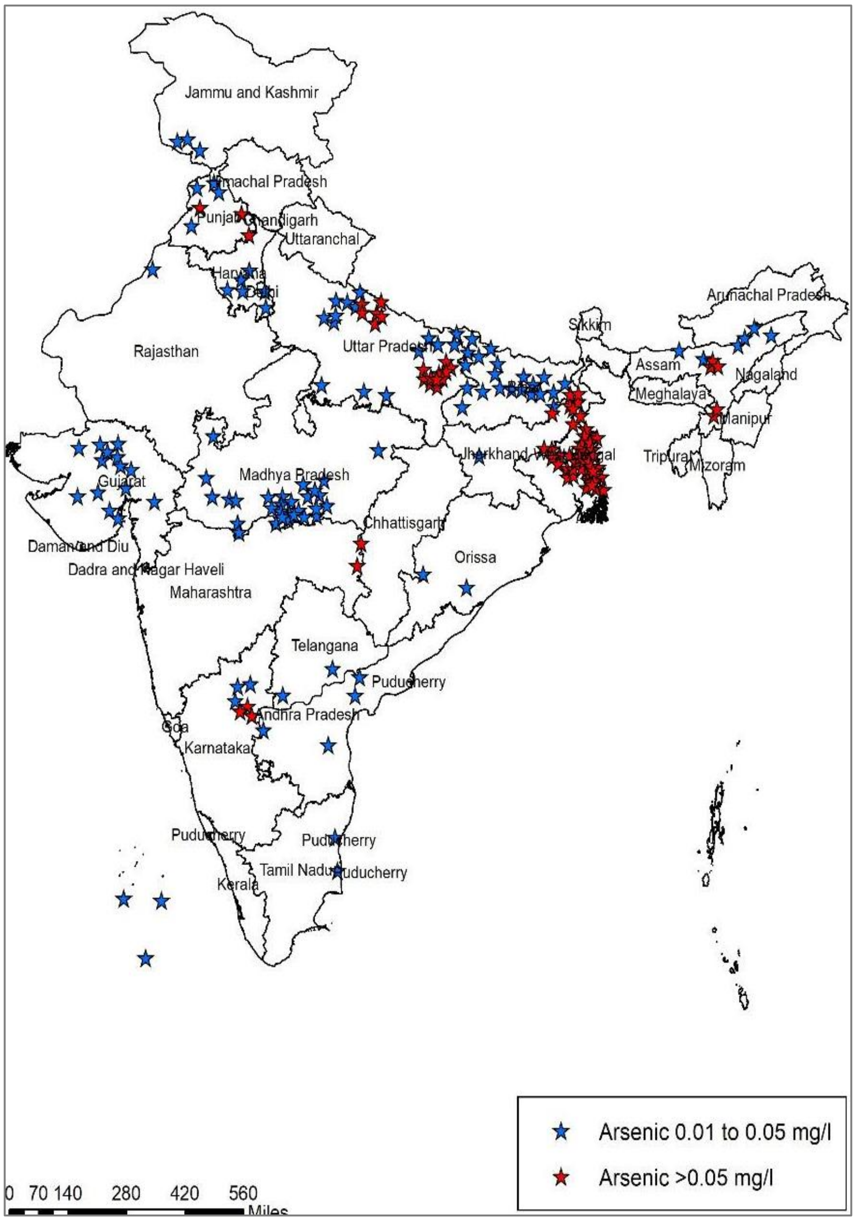


Figure 10.5: Major locations of Arsenic in Ground Water
(**Source:** *CGWB.*)

Arsenite is largely non-charged at pH below 10.2. This reduces the availability of the trivalent form of arsenic for precipitation, adsorption, or ion exchange. Hence, most available removal procedures are more efficient for arsenate. As a result, treatment technologies that use a two-step strategy consisting of early oxidation from arsenite to arsenate followed by an arsenate removal process are thought to be more effective. The approaches currently available for removing arsenic from water are summarised in Figure 10.6.

Figure 10.6: Arsenic remediation methods

Sustainability of the processes can be enhanced by online monitoring and appropriate residue management and reuse.

10.9.1 Arsenic Removal Techniques

10.9.1.1 Removal by Oxidation

The soluble arsenite is converted to arsenate by oxidation. As this does not remove arsenic from the solution, another removal process, such as adsorption, coagulation, or ion exchange, is also required. As arsenite is the predominant form of arsenic at near-neutral pH, especially when water is extracted from the ground, oxidation is a critical phase for anoxic groundwater. Many chemicals, ambient oxygen, as well as microbes, have already been employed to directly oxidise arsenite in water.

The most often utilised oxidants in developing countries are atmospheric oxygen, hypochlorite, and permanganate. Arsenite oxidation with oxygen is a very slow process that can take hours or weeks to complete. Chemicals, on the other hand, such as chlorine, ozone, and permanganate, can rapidly oxidise As^{3+} to As^{5+}. Despite this improved oxidation, interfering chemicals in water must be addressed when selecting the appropriate oxidant, as these substances can significantly alter and determine the kinetics of As^{3+} oxidation. Furthermore, this requires a sophisticated treatment that yields arsenic-containing residue at the end that is difficult to dispose of. Thus, to effectively remove arsenic from a solution by oxidation, oxidants must be properly chosen.

10.9.1.2 Removal through Coagulation-Flocculation

Coagulation and flocculation are two of the most widely used and well-documented processes for removing arsenic from water. Positively charged coagulants [e.g., aluminium sulphate ($Al_2(SO_4)_3$), ferric chloride ($FeCl_3$)] lower the negative charge of colloids causing them to collide and grow larger during coagulation. The addition of an anionic flocculant, on the other hand, produces bridging or charge neutralisation between the generated bigger particles, resulting in the development of flocs. During these processes, the chemicals convert dissolved arsenic into an insoluble solid, which then precipitates. Alternatively, soluble arsenic species can be co-precipitated by incorporating them into a metal hydroxide phase. In either case, solids can be removed later using sedimentation and/or filtration.

The efficacy of different coagulants in removing arsenic varies with pH. Under pH 7.6, $Al_2(SO_4)_3$ and $FeCl_3$ are both effective at removing arsenic from water. Most studies have found that arsenate is more efficiently eliminated than arsenite and that $FeCl_3$ is a superior coagulant than $Al_2(SO_4)_3$ at pH greater than 7.6. Despite their claimed decreased efficiency when compared to ferric chloride, aluminium-based coagulants were nevertheless capable of lowering arsenic concentrations below the acceptable maximum concentration level (MCL) of 10 µg/L.

The major drawback of coagulation-flocculation is the production of high amounts of arsenic-concentrated sludge. The management of this sludge is necessary to prevent the consequence of secondary pollution of the environment. Moreover, the treatment of sludge is costly. These limitations make this process less feasible, especially in field conditions.

10.9.1.3 Reduction, Coagulation & Filtration

An effective alternative to using bulk ferric chloride as an arsenic treatment reagent is to introduce ferric ions into the treated water stream in-situ by the controlled electrolytic dissolution of iron metal precursor (anode). These sacrificial electrodes corrode under controlled conditions to generate the reagent (ferric ion) on demand within a side stream of the treated flow. Then, the freshly generated reagent is re-injected into the treated flow to maintain the required coagulant dose and pH.

This electrolytic approach to ferric reagent generation is based on Faraday's laws of electrolysis. It is an accurate, and safe approach that conforms to Indian Standards. The modular design of the electrolytic generator allows a flexible treatment system scaling from as low as 20,000 LPD to 10 MLD. The compact electrolytic system integrates into existing treatment systems infrastructure and also can be retrofitted to replace bulk ferric solutions. The on-demand generation of electrolytic reagent has multiple benefits over bulk ferric salt solutions:

- The iron precursor used in the electrolytic process is certified and broadly available that minimises impact of reagent co-contaminants on treated water quality;
- Electrolytic reagent is less expensive than bulk ferric reagent due to the lower cost of consumables required for its generation;
- Fully automated system controls both ferric reagent dose and treated water pH during the arsenic treatment process. Such a capability allows to achieve lower arsenic levels in the effluent under optimal ferric reagent dose and pH;
- Optimisation of ferric dosing reduces entire material use, sludge volumes, filter backwash frequency, and water loss;
- Integration of online real-time monitoring contributes to entire system automation and optimisation. Monitoring permits the potential reuse of a proportion of backwash water thereby further reducing water loss;

- On-site reagent generation lowers greenhouse gas generation, supply chain risks, and logistical challenges associated with bulk ferric salts.

10.9.1.4 Adsorption

Adsorption is a process that uses materials as a medium for the removal of substances from gases or liquids. Substances are separated from one phase and accumulated on the surface of the adsorption medium. Van der Waals forces and electrostatic interactions between adsorbate molecules/ions and adsorbent surface drive this process. As a result, it is critical to initially characterise the adsorbent surface features (e.g., surface area, polarity) before using them for adsorption. Adsorption-based removal of contaminants is more sustainable as the process of adsorption does not need energy. The process of water purification itself can be environmentally friendly if the materials are green and if the used adsorbent do not cause environmental impact.

Activated carbon, coal, red mud, fly ash, animal-derived products such as bone-char, kaolinite, montmorillonite, goethite, zeolites, activated alumina, titanium dioxide, iron hydroxide, zero-valent iron, chitosan, and cation-exchange resins have all been explored in various field studies. Iron-based adsorption is a new approach for treating arsenic-contaminated water. This can be explained by the fact that inorganic arsenic species have a strong affinity for iron. Arsenic can be removed from water by iron-based materials serving as a sorbent, co-precipitant, or contaminant-immobilizing agent, or by functioning as a reductant.

Adsorption has been recognised as the most extensively used approach for arsenic removal due to various benefits such as relatively high arsenic removal efficiency, ease of operation and handling, cost-effectiveness, and no sludge creation. However, the adsorption of arsenic strongly depends on the system's concentration and pH. At low pH, arsenate adsorption is favoured, whereas, for arsenite, maximum adsorption can be obtained between pH 4 and 10. Furthermore, contaminated water contains more than just arsenic, it is always accompanied by other ions, such as phosphate and silicate, which compete for adsorption sites. Aside from the system parameters, the type of adsorbent itself can reduce the effectiveness of adsorption in arsenic removal. A variety of adsorbents, including those listed above, have already been investigated for the removal of arsenic from water. Most traditional adsorbents, on the other hand, have irregular pore architectures and poor specific surface areas, resulting in limited adsorption capabilities. Lack of selectivity, relatively weak interactions with metallic ions, and regeneration challenges can also limit these sorbents' capacity to lower the arsenic concentrations to levels below the maximum concentration threshold.

These limitations of traditional adsorbents have led to the development of new materials, especially nanomaterials.

10.9.1.5 Ion Exchange

Synthetic ion exchange resins can be used for arsenic removal. Generally, these resins have a base of cross-linked polymer matrices to which charged functional groups such as amine or quaternary ammonium are attached. Ion exchange technologies are mostly used with water that has low TDS and where arsenic is the only significant contaminant. Their exchange capacity depends on the composition of the groundwater and the influence of interfering anions like sulphates, phosphates, silicates, and nitrates. Hence, ion exchange can be performed after other treatment processes such as coagulation. Various ion exchange resins for arsenic removal are loaded with TiO_2, zirconium oxide, iron oxides, or MnO_2. While ion exchange is useful, the contaminants must be in the ionic form and should be exchangeable with ions on the resin; however, this is not the case with neutral species. Resins loaded with iron oxides and oxyhydroxides have also been used to remove such species.

10.9.1.6 Application of Nanomaterials for the Removal of Arsenic from Water

Advances in nanoscience and nanotechnology have paved the way for the development of various nanomaterials for the remediation of contaminated water. Due to their high specific surface area, high reactivity, and high specificity, nanoparticles have been given considerable environmental attention as novel adsorbents of contaminants, such as heavy metals and arsenic, from aqueous solutions. Carbon nanotubes and nanocomposites, titanium oxide-based nanoparticles, iron-based nanoparticles, and other metal-based nanoparticles are among the most commonly researched nanoparticles for arsenic-contaminated water treatment. Many of these materials have also been modified with other metal oxides, such as zirconia, to improve the adsorption capacity. Although the literature may refer to them as nanoparticles, the materials used in the field are nanostructured materials in the form of beads of millimetre scale particles. A typical iron and arsenic removal community WTP in Punjab in shown in Figure 10.7.

Figure 10.7: A typical iron and arsenic removal community WTP in Punjab. The blue-coloured unit, below the overhead tank, is the purification unit and the treated water is distributed to the village.
(Source: *IIT, Madras.*)

One of the most notable nanoscale systems currently used for the efficient removal of arsenic is metastable two-line ferrihydrite, which adsorbs both arsenite and arsenate in natural pH conditions with excellent selectivity and in presence of the typical interfering ions present in natural waters. The technology is often referred to as Anion and Metal Removal by Indian Technology (AMRIT). The specific adsorbent used is confined metastable 2-line ferrihydrite (CM2LF) in biopolymer cages. Its arsenic uptake capacity is about 25 to 30 mg g^{-1} in field conditions. Due to the large adsorbate capacity, the replacement of adsorption media is less frequent. The arsenic laden waste does not release arsenic in the ambient conditions of soil and it can be disposed off safely, following protocol. As a result of the lower cost, high efficiency of removal of arsenite and arsenate, safe disposal of the spent media and reduced maintenance cost, this method has been applied in various parts of the country. Besides, the media can be used for both domestic and community filtration. In cases where local sources are contaminated by other contaminants, such as uranium, the method is found to be effective. Although a reactivation protocol exists to regenerate the arsenic-saturated CM_2LF, given the high-adsorption capacity, the frequency of adsorbate replacement is as long as 2 to 3 years. In addition, reactivation produces sludge and its management is a concern.

A block diagram of a nanotechnology-enabled water purification plant for arsenic and iron removal is shown in Figure 10.8. Such integrated plants are desired in most places as both the contaminants exist together. The input water after initial oxidation is passed through iron and arsenic removal units in succession. Any other contaminant present may be removed by a polisher unit. Post-chlorination ensures the removal of microbes. The system may be fitted with sensors and devices at various locations for monitoring and control.

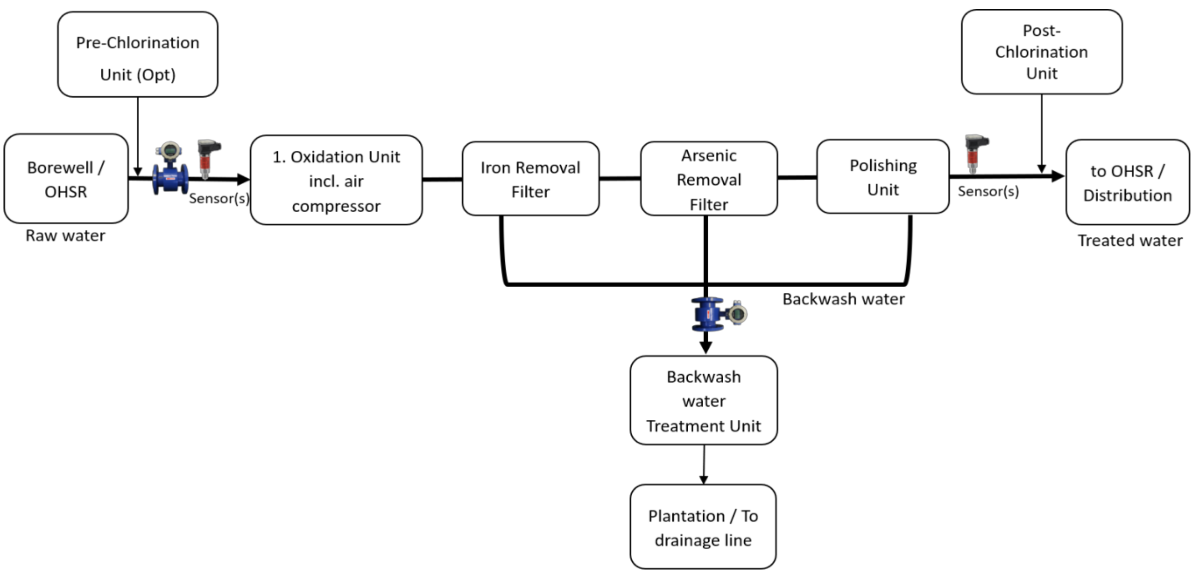


Figure 10.8: Block diagram of a nanotechnology-enabled water purification plant for arsenic and iron removal

10.9.1.7 Advanced Plants with Integrated Sensors

Integration of sensors along with water purification devices and treatment plants can promote substantial advancement in environmental water monitoring and industrial wastewater management. On-site and continuous monitoring of water quality can be performed using portable sensors, enabling adherence to effluent regulations and safety parameters as well as for the evaluation of efficiency of the treatment process. The entire plant can be Internet of Things (IoT)-enabled in terms of equipment automation and water analysis. Implementation of IoT-enabled plants can allow authorities to obtain real-time qualitative and quantitative information about the plant's operation over a long-time window. Such an advanced water treatment/purification plant will also be beneficial for public awareness and establishing regulations for water quality. Figure 10.9 shows a block diagram for such an advanced plant for arsenic removal.

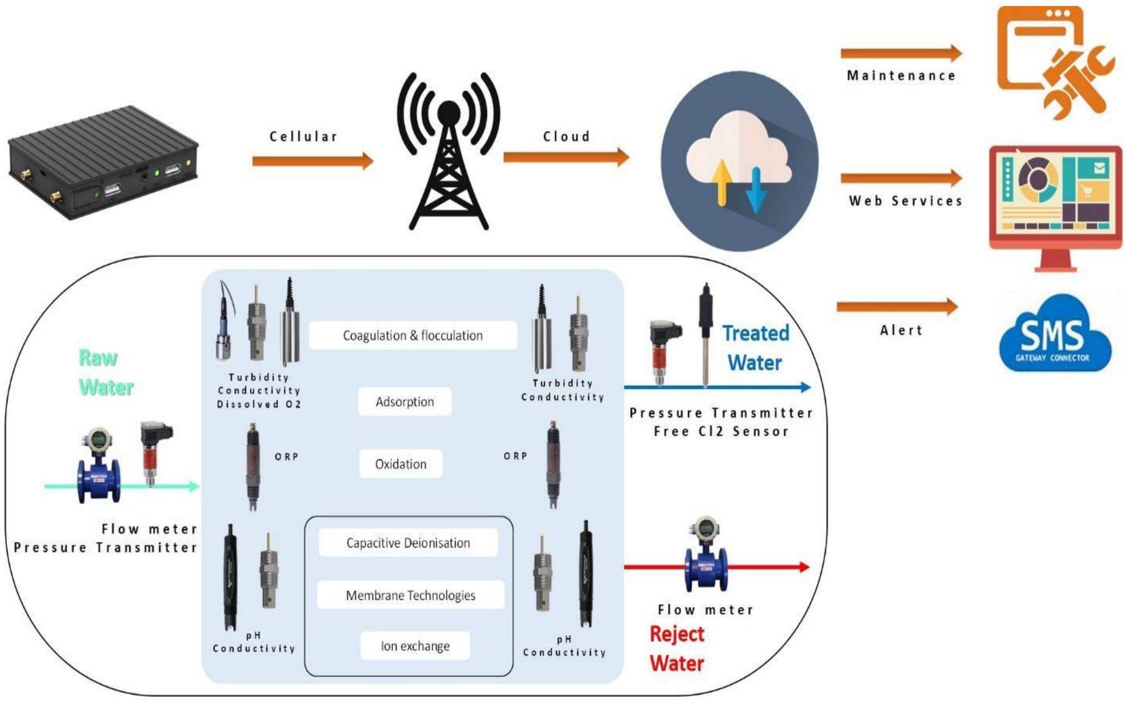


Figure 10.9: IOT-based monitoring of various arsenic removal technologies

Data from the treatment units on the time duration of operation, water dispensed, voltage at the pumping station, current used by the pump, and water quality in terms of TDS are collected and presented using a central dashboard. The data may be available and displayed at multiple locations and may be provided to the pump operator/user community in multiple ways. This ensures that all the treatment plants are used effectively and problems, if any, are addressed. Such problems may relate to the health of the motor, electrical supply, availability of the operator, clogging, efficiency of filtration, etc. More sensors and better controls can also be put in place and this has been implemented in select locations.

10.10 Reject/Residue Management of Arsenic, Fluoride, and Iron Removal Plants

In the preceding sections, various treatment techniques have been described for the removal of arsenic, fluoride, and iron from raw groundwater to produce potable water. These treatment plants generate hazardous liquid and solid residues/rejects containing a high concentration of arsenic/fluoride/iron posing a serious threat of contamination of groundwater/surface water/land if not properly and safely disposed of. The waste stream in arsenic, fluoride, and iron removal plants can be categorised as follows:

a) Backwash wastewater stream;
b) Sludge from treatment reactors;
c) Spent adsorbents/nanomaterials and replaceable catalysts.

The above rejects/residue need to be treated and safely disposed of to prevent contamination of groundwater/surface water/land. The residue/rejects from arsenic/fluoride removal plants are more hazardous/toxic as compared to that of iron removal plants, but IRPs produce more quantity of sludge.

The following residue/reject management should be employed:

- Backwash water generated from various techniques of arsenic, fluoride, and iron removal plants shall be analysed on a weekly basis.

- The backwash water shall be treated in the sedimentation tanks. A sedimentation tank for backwash water is designed based on the total backwash water generated with the provision of sludge produced. The sedimentation tank may be rectangular or square type with a side water depth of 1.2 to 1.5 m with a free board of 0.3 m. The sludge storage facility is designed for 13 to 15 years or based on the availability of land with a rectangular or square shape. A geo-membrane liner shall be used to restrict the percolation of leachate in the ground.
- For arsenic and fluoride removal plants, the supernatant of sedimentation tanks shall satisfy the effluent standards prior to disposal as per MOEFCC (2016). The supernatant of the sedimentation tank shall satisfy effluent standards of maximum permissible limit (mg/L) as given in Table 10.3 below.

Table 10.3: Permissible Limit of As, F, and Fe (mg/L) for Supernatant of Sedimentation Tanks

Parameters	Into inland surface water	On land for irrigation
Arsenic	0.2	Not permitted
Fluoride	2.0	2.0
Iron	3.0	3.0

- Sludge dewatering shall be carried out by sand drying beds, freeze assisted sand beds, vacuum assisted beds, solar drying beds, etc.
- Sludge may be used as building construction materials like bricks, cement, etc.
- Engineered landfills, if necessary, shall be designed and provided for arsenic, fluoride, and iron removal for a period of 15 years or the period decided as per the availability of land.
- Exhausted/spent media of arsenic, fluoride removal plants shall be tested for the toxicity characteristics leaching procedure (TCLP) prior to transport to the designated disposal facility.
- Supernatant after sedimentation and dewatering shall be tested before discharging into the environment.
- Sedimentation tank and sludge storage facility shall be included as a mandatory part of the WTP.
- The sludge shall satisfy standards for restricted landfills or will be used as construction materials with the specified standards. The long-term solution is to recycle the sludge and use it for beneficial purposes. One of the techniques which is adopted to treat hazardous/toxic waste is to solidify it, which results in stabilisation of the components of waste.

Handbook on Drinking Water Treatment Technologies (2023) published by the Department of Drinking Water and Sanitation, Ministry of Jal Shakti, Government of India, should be referred for additional arsenic removal technologies.

10.11 Removal of Nitrate

Nitrates are inorganic nitrogen and oxygen compounds that exist naturally and synthetically in the environment. Nitrates are found in the earth's atmosphere, soil, and water. They are easily biodegradable and highly soluble in water. Plant breakdown, animal waste, and agricultural by-products all produce nitrates. Rainwater, floods, and soil erosion can all contribute nitrates into groundwater supplies.

Water containing less than 45 mg/L (acceptable limit as per IS 10500:2012) of nitrate (as NO_3) is safe to consume. Long-term exposure to high nitrate concentrations can cause major health problems in children, such as methemoglobinemia, popularly known as blue baby syndrome, and stomach cancer in adults. A child with blue baby syndrome will start displaying bluish or brownish colours around their lips and mouth, nose, and nail beds. It can also present flu-like symptoms, such as nausea, vomiting, and diarrhoea.

Nitrates enter the water supply via agricultural runoff, industrial waste, fertiliser, and herbicide use, leaking septic tanks, and broken sewage systems. Naturally occurring microorganisms in soil can also metabolise nitrogen, resulting in nitrate deposits in the ground. Manure from farmlands can be carried into streams and lakes by rainwater. Animal excrement is another prominent source of nitrate contamination in water systems. Crop irrigation can deeply embed nitrates in the soil, and as precipitation percolates down through the earth, the nitrates can enter aquifers. Increased amounts of nitrates in underground water storage utilised for drinking water supply systems can occur as natural deposits of nitrates erode.

Many districts in Andhra Pradesh, Bihar, Delhi, Haryana, Himachal Pradesh, Karnataka, Kerala, Madhya Pradesh, Maharashtra, Orissa, Punjab, Tamil Nadu, Rajasthan, West Bengal, and Uttar Pradesh have high nitrate concentrations (greater than 45 mg/L) (Figure 10.10). The highest value was 3080 mg/L found in Bikaner, Rajasthan.

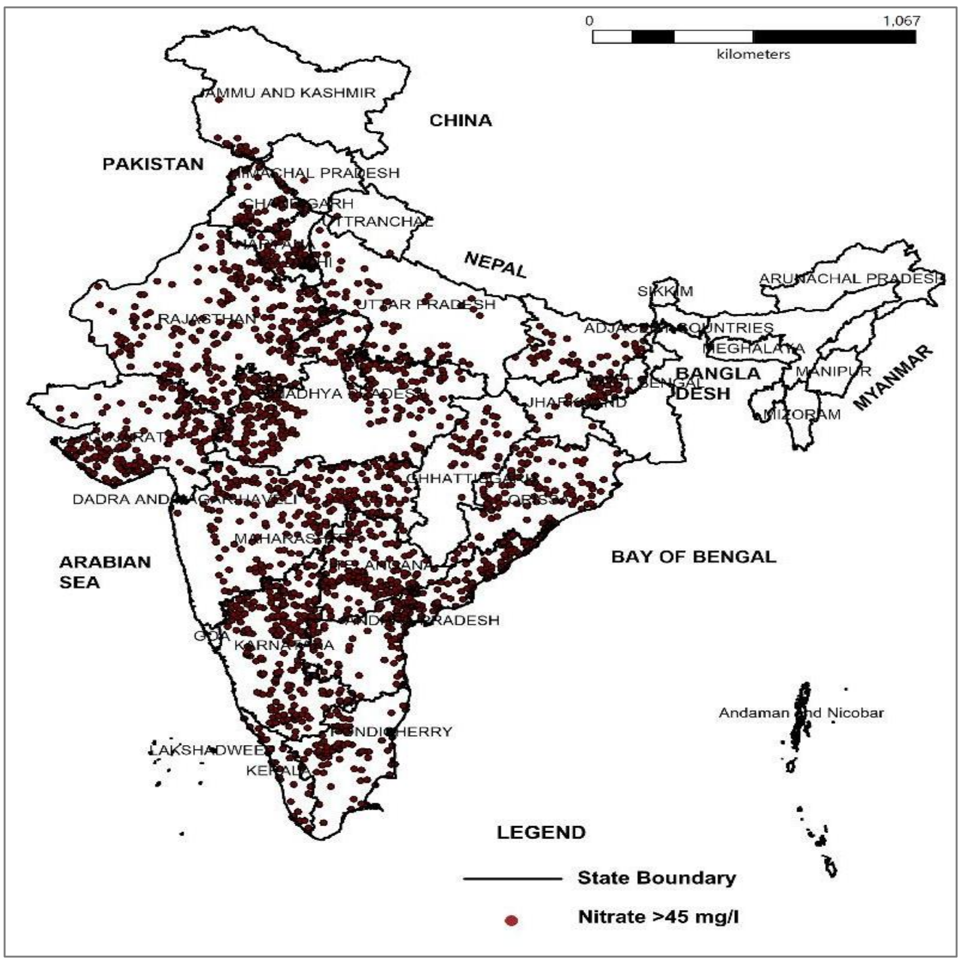


Figure 10.10: Location of Nitrate in Groundwater

10.11.1 Techniques for Removal of Nitrates

Nitrates in water can be removed via reverse osmosis (RO), distillation, or ion exchange resin. Nitrates are notoriously difficult pollutants to remove from water. Nitrates will not be removed by sediment filters, carbon filters, or a UF system's hollow fibre membrane. A typical ion exchange water softener, meanwhile, will neither reduce nor remove nitrates.

a) Reverse osmosis

RO eliminates pollutants by pushing pressured water across a semi-permeable membrane with small pores.

b) Ion exchange

Passing water through an ion exchange process is one of the most effective ways to remove nitrates. These nitrate removal devices, however, use chloride ions rather than the sodium ions normally used in traditional water softeners. When polluted water travels through the tank or cartridge, the resin beads capture the nitrate ions and replace them with innocuous chloride ions. Because both chloride and nitrate have a negative charge, this is known as anion exchange (making them anions). They will only be capable of processing a certain volume of water before all the chloride ions are exhausted and the media will either need to be regenerated or the cartridge replaced.

For backwashing the system, it will periodically need to flush the system with sodium chloride to regenerate the resin and flush out a brine solution. Because this brine solution contains nitrates, it should not be discharged into any natural water source or where animals could get it.

Handbook on Drinking Water Treatment Technologies (2023) published by the Department of Drinking Water and Sanitation, Ministry of Jal Shakti, Government of India, should be referred for additional nitrate removal technologies.

10.12 Uranium

Uranium is a naturally occurring radioactive element that occurs in low concentrations in nature. It is present in certain types of soils and rocks, especially granites and metasedimentary rocks. Uranium concentrations in groundwater in shallow aquifers across the country range from 0.0 to 2876 µg/L, suggesting that groundwater uranium concentrations vary by several orders of magnitude. Punjab (where 24.2% of groundwater samples have been detected to have uranium concentrations greater than the limit prescribed in IS 10500), Haryana (10.6% samples), Telangana (10.1% samples), Delhi (11.7% samples), Rajasthan (7.2% samples), Andhra Pradesh (4.9% samples), and Uttar Pradesh (4% samples) are the most affected states (Figure 10.11).

The radioactivity of naturally occurring uranium is extremely low. The chemical properties of uranium in drinking water, however, are far more concerning than its radioactivity.

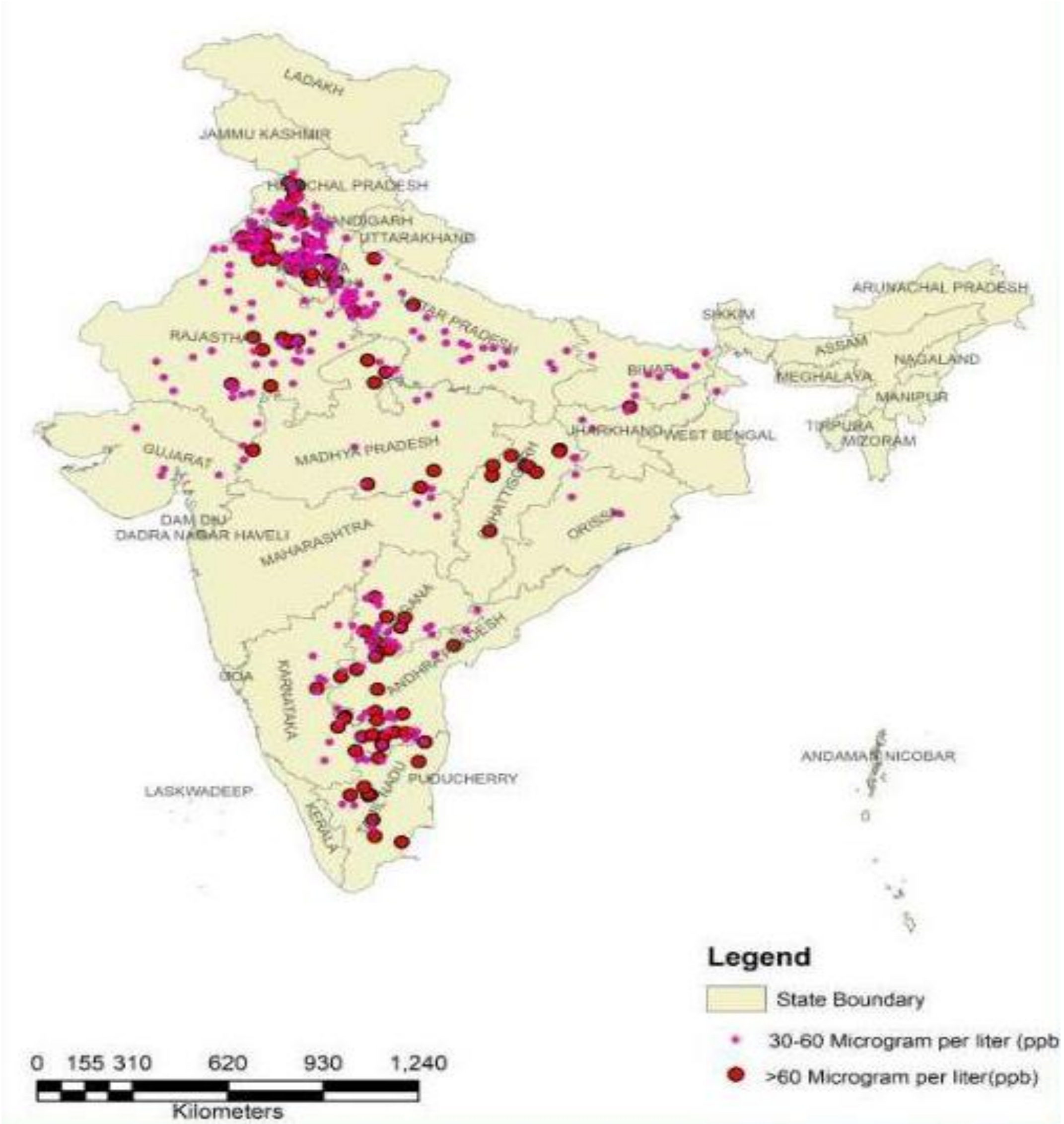


Figure 10.11: Location of High Uranium in Groundwater
[**Source:** *NDC CHQ* (8/4/2020).]

10.12.1 Remedial Measures

Ex-situ treatment of radioactive contaminants in groundwater falls into the following categories:

(i) **Adsorption or ion exchange:** The water-soluble contaminants are captured by sorption onto solid support that can be natural or synthetic material.
(ii) **Reactive sorption:** The process is based on the reaction of contaminants with a solid substrate. It is often applied in-situ as a barrier wall through which the contaminant is forced through and trapped.
(iii) **Nanomaterials**: Nanomaterials have been found to be effective in removing uranium in ground water. Several such community plants are working in Punjab at present. In several places, other contaminants such as arsenic and iron are found to co-exist with uranium.
(iv) **Precipitation:** This process is mostly practiced above ground and involves the addition of alkali to raise the pH and precipitate the oxide or hydroxide.
(v) **RO:** In this process, water is transported through a high-pressure gradient through a membrane essentially non-permeable to the contaminant.
(vi) **Stripping:** Only applicable to volatile contaminants like radon.

Remediation solutions based on in-situ chemical stabilisation are only as effective as the site's geochemistry allows. Such chemical technologies may be generally grouped according to the following paradigm:

(i) **Redox Technologies:** These technologies attempt to manipulate oxidation-reduction conditions of the subsurface to reduce uranium to uranous (uranium IV) forms. The techniques include in-situ redox manipulation using sodium dithionite, zero-valent iron, microbial-induced reduction, and calcium polysulfide technologies. The common deficiency of technologies in this category is that the reduced environment and corresponding uranium precipitate are easily re-oxidised over time. Consequently, over time the "treated" uranium remobilise in the system.
(ii) **Co-precipitated Iron Oxy-hydroxide:** This technology affects only temporary stabilisation because the reaction is reversed as the precipitate ages.
(iii) **Phosphate Precipitation Technologies**: These techniques use and alter phosphate with uranyl (uranium VI) forms to remove soluble uranium and prevent additional uranium dissolution by sequestration, immobilisation, or precipitation. The resulting reaction seeks to create a stable, long-lasting reaction that removes the source of ongoing uranium contamination in the groundwater. However, this group of technologies requires further development.
(iv) **Flushing Technologies:** This category of remediation procedures employs a variety of leaching solutions to dissolve solid-phase uranium and hydraulic extraction techniques to remove the dissolved uranium. Because of subsurface stratigraphic heterogeneities, thorough treatment is difficult to achieve. Hydraulic capture and capture of the mobilised uranium can be problematic.
(v) Groundwater with a greater uranium percentage can be made potable using procedures like RO. Based on a field study carried out in Punjab, it was established that uranium content in RO treated water is below 0.1 µg/L. However, rejecting water disposal of RO is an issue to be taken care of.
(vi) Spent media and discharges from the units must be handled as discussed in section 10.10.

10.13 Removal of Ammonia

Ammonia is utilised in the production of fertilisers and animal feed, as well as the production of fibres, plastics, explosives, paper, and rubber. It is used as a coolant, in metal processing, and as a starting product for many nitrogen-containing compounds. Ammonia and ammonium salts are used in cleansing agents and as food additives, and ammonium chloride is used as a diuretic. On dissolution in water, ammonia forms the ammonium cation. Hydroxyl ions are formed at the same time. In natural levels of ammonia in groundwater are usually below 0.2 mg/L. Higher natural contents (up to 3 mg/L) are found in strata rich in humic substances or iron or in forests. Surface waters may contain up to 12 mg/L. Ammonia may be present in drinking water as a result of chloramine disinfection. Ammonia levels above geogenic levels are a major indicator of faecal pollution. If drinking water contains more than 0.2 mg/L of ammonia and is chlorinated, taste and odour problems, as well as lower disinfection efficacy, are to be expected, since up to 68% of the chlorine may react with the ammonia and become unavailable for disinfection.

Excessive ammonia in drinking water can cause nitrification in the water distribution system, leading to many problems including corrosion, aesthetic issues, and pH decrease.

10.13.1 Remedial Measures

(i) Breakpoint chlorination is an effective way to remove ammonium from drinking water. It has a low spatial requirement, non-sensitivity to temperature variations, and adaptability to existing facilities. However, it can result in excessive chlorine consumption and the development of harmful chlorinated by-products in drinking water.

(ii) In catalytic oxidation technology, iron–manganese co-oxide filter film (MeO) can remove ammonium from drinking water by chemical catalytic oxidation. Catalytic oxidation is a new way to remove high concentrations of ammonia from drinking water.
   a) Quartz sand coated with metal oxide filter film (MeO) (Generally, iron–manganese co-oxide filter film) can remove ammonium from drinking water by chemical catalytic oxidation.
   b) It is more resistant to low temperatures and requires less time to start up than other bio filtration methods.

(iii) Various other studies are also being carried out for the removal of ammonia from raw water for drinking water supply with concentration much higher.

10.14 Demineralisation of Water

Conventional methods of water treatment do not materially change the mineral content of water. Base exchange softening merely converts the calcium and magnesium salts to the corresponding sodium salts. Lime softening causes a slight decrease in the contents of total solids but does not bring about any decrease in the content of sodium chloride or sulphate. Hence, these methods are not effective in converting brackish water into a potable one. For providing a potable supply in a brackish water area, the least mineralised water source could have been prospected. When potable water is unavailable, some method of treatment has to be adopted.

10.14.1 Distillation

Of the processes of removing water from saline solutions, distillation is the oldest and in terms of established plants, the most productive. It differs from the other processes by its passage of water through the vapour phase. The plant design is directed at tapping the most economical source of heat energy and exploiting the most efficient processes of heat transfer. While relatively small quantities of water are to be distilled, straight or single-effect distillation is preferred because of the simplicity of operation and the lower capital cost of the installation. With larger outputs, improvement in efficiency acquires much greater importance because of the much higher rates of evaporation involved and the need for highly efficient heat transfer systems. Problems of scale formation also play a significant role.

The performance of an evaporator plant is measured by the specific heat consumption, i.e., the number of kilocalories required to produce 1 kg of distillate. Distillation plants are generally better for lower values of specific heat consumption. The introduction of the flash evaporator has helped in better economics of heat recoveries and more efficient plants can be built more cheaply. It is only in such situations where natural gas or fuel is available cheaply that low thermal performance evaporators can be used with the resultant saving in capital cost.

10.14.2 Solar Stills

Solar energy can be harnessed with the help of a system of mirrors following the path of the sun to focus the sunlight on sheets of water. In one of the popular methods, the saltwater trickles down to trays mounted on an inclined compartment provided with glass sides and a heat-insulated base that screens the condensing chamber from the sun. Since the focusing mirrors form an important element in the cost of the stills, the development of cheaper non-focusing types of mirrors and the use of inexpensive construction materials have been resorted to. In basin solar stills, a commonly used design, saltwater tanks, filled either by gravity or by stainless steel impeller pumps, feed the solar still whose cover is at a shallow angle of 10° to 18° with the glass panes tightly sealed to the holding frame and the joints between the still cover and the vertical walls perfectly tight. The collecting troughs at the foot of the still cover must be built in such a way that water can freely drain to the pipe carrying the distillate to the freshwater tank while preventing any contaminated water from entering from the roof or ground on which it is built. In addition to the freshwater tank, it is a good practice to construct additional distilled water storage to balance out the fluctuations between production and demand.

The best situations for the use of solar distillation are the isolated areas and certain regions where freshwater is unobtainable, solar intensities are high, fuel resources are meagre, and industrial development is poor. Hence, this process can rarely be used.

10.15 Membrane Processes

Certain natural and synthetic membranes have the property of permitting the solvent (water) to get through them but not the solute. Such semi-permeable membranes permit the separation of solute from solvent. This phenomenon is known as osmosis. The membranes are categorised based on pore size as microfiltration (MF), Ultrafiltration (UF), Nano filtration (NF) and RO as described below:

10.15.1 Microfiltration

MF is a pressure-driven separation process that is commonly used to concentrate, purify, or separate macromolecules, colloids, and suspended particles from solution. Nominal pore diameters for MF membranes are typically in the range of 0.1 to 1.0 μm. Because of the vast range of pore size, MF membranes can be utilised in a variety of applications that need the separation of viruses, bacteria, aerosols, and other macromolecules from liquids. MF can be performed in two modes: dead-end (inline) filtration and cross-flow (tangential flow) filtration. The predominant flow direction in dead-end filtration is perpendicular to the membrane. The suspended particles are constantly drawn towards the membrane, where they settle on the surface or inside the membrane pores. Particle deposition results in a rising resistance to flow and, as a result, a decreased permeate flux rate. To decrease deposition, MF is frequently performed in the cross-flow mode (tangential flow), with the primary flow direction tangential to the membrane.

10.15.2 UF Membranes

UF is the most commonly used membrane-based water treatment process. Solutes/particles of a size of 0.03 microns and larger are rejected by UF membranes. Particles and microorganisms are removed using this process; however, ions are not. UF creates water with very high purity and low silt density by providing a pressure-driven barrier to suspended particles, bacteria, viruses, endotoxins, and other pathogens. Hydrostatic pressure presses a liquid against a semi-permeable membrane. UF employs hollow fibres of membrane material, with feed water flowing either within the shell or through the fibre lumen. Suspended solids and high molecular weight solutes are trapped, whereas water and low molecular weight solutes pass through. The UF has consistently treated water quality, low land requirements, and intensive automation. Moreover, it can eliminate microorganisms from water including viruses. UF technology can now offer better processing efficiency, greater treatment effect, and even lower energy usage as compared to conventional water treatment processes. The service life of UF membranes ranges from three to five years or longer. Hollow-fibre, tubular, spiral-wrapped, and plate and frame are commercially available in India for water treatment. The UF process has a greater removal rate of turbidity and particulate matter than the traditional process; effluent turbidity is constant below 0.1 NTU and particle removal is up to 99.9%.

10.15.3 Nano Filtration (NF)

NF membranes are presently used mostly for industrial applications, as stated above. Main applications are hardness removal, salt recovery, colour separation, applications in Food and

Beverage industries for juice concentration etc. Since their use is limited to small flows compared to Municipal requirements, present prices are on higher side.
Many other technologies such as Lime soda softening, resin-based softening (in few cases), Reverse Osmosis (especially for Brackish and Sea water) are available for larger flows and at very competitive OPEX and CAPEX levels, it would take some time for NF to find a sweet spot in direct Municipal applications.

Details of MF, UF, and NF are provided in Figure 10.12.

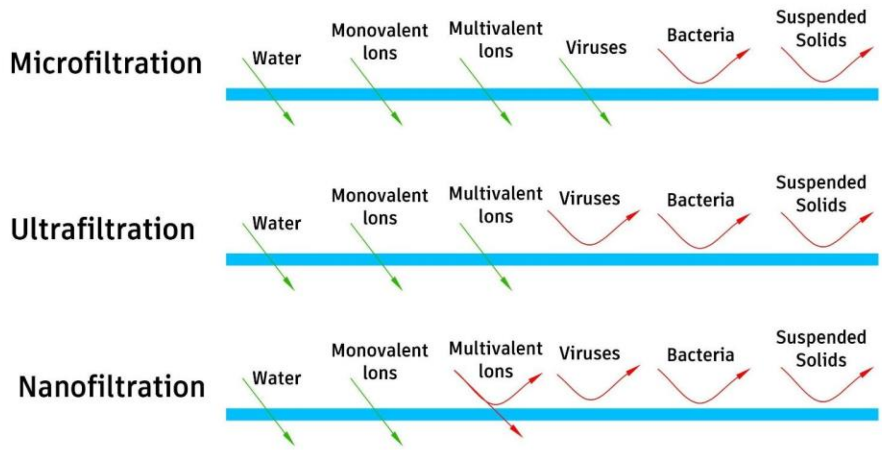


Figure 10.12: Membrane Process Characteristics

10.15.4 Reverse Osmosis (RO)

RO is a membrane permeation process for separating relatively pure water (or other solvents) from a less pure solution. Osmosis is a process of water diffusion through a semi-permeable membrane from a low TDS water to water having high TDS until both sides of the semi-permeable membrane have an equilibrium. In equilibrium conditions, the osmotic pressure of water is equal to the pressure difference on either side of the membrane. The potential gradient of TDS across the membrane causes the primary water flow. If the applied pressure on the membrane is more than the osmotic pressure, water passes through membrane pores on permeate side, while the high TDS solution remains on the other side of the membrane. This process is commonly known as “reverse osmosis” since it is the reverse of normal osmosis. The solution is passed over the surface of an appropriate semi-permeable membrane at a pressure exceeding the effective osmotic pressure of the feed solution (Figure 10.13). The permeating liquid is collected as the product and the concentrated feed solution is generally discarded. The membrane must be highly permeable to water, highly impermeable to solutes, and capable of withstanding the applied pressure without failure. Because of its simplicity in concept and execution, RO appears to have considerable potential for a wide application in water and used water treatment.

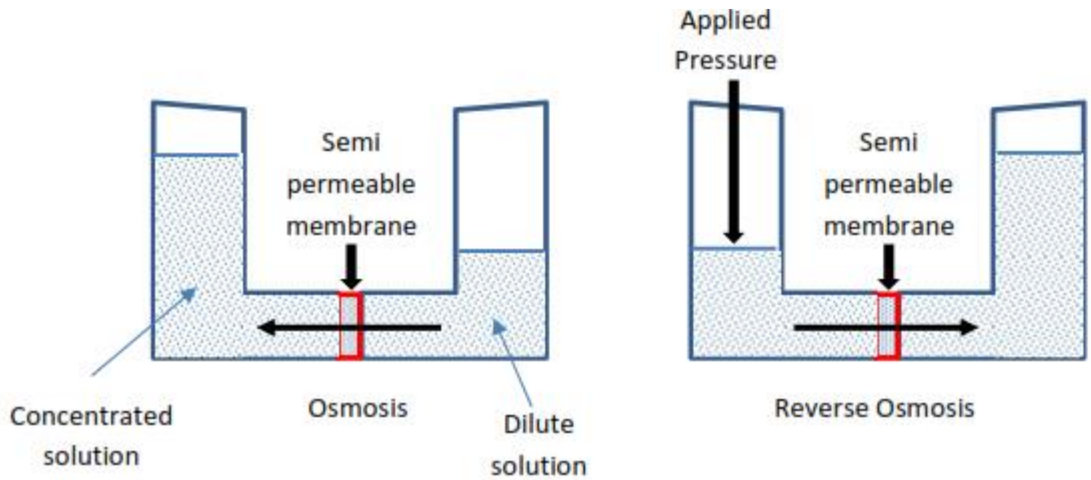


Figure 10.13: Reverse Osmosis (RO) Process

There are several materials used in manufacturing of RO membranes. Cellulose acetate was the first membrane material. Polysulfone having aromatic polyamides coating is very popular currently, and is also referred as thin film composite (TFC) membranes. RO membranes can literally stop all the ions present in the water making water highly corrosive. Hence, it is ensured that the treated water has sufficient quantity of TDS to avoid production of corrosive water.

10.15.5 Electrodialysis (ED)

Unaided osmosis is a relatively slow process and, hence, attempts have been made to combine it with electrolysis. The application of an external electromotive force can draw the ions away from the salt solution towards the electrodes so that the solution is impoverished of its salt content. The reunion of the ions by diffusion can be prevented by using suitable membranes to separate the cathode and anode chambers and also by continuously removing the relatively concentrated solution of the electrolytes from the electrode chambers. To obtain purification of sufficient magnitude, several electrolytic cells have to be used in series. In essence, the apparatus would consist of several electrolytic cells, each of which is composed of three compartments separated from each other by suitable membranes. The saline water circulates in series through the middle compartments of the cells and undergoes progressive purification. The number of cells and the rate of flow may be adjusted to give the degree of purification required. A direct current of 110 to 220 V is employed. The electrodes are continuously washed with the treated water. One of the main disadvantages of the ED process is that the membranes get badly damaged as a result of corrosion and scale formation. Another disadvantage is that the cost goes up steeply as the total solids content of the finished water decreases. Power loss is minimised if the water is demineralised only partially to final concentrations of less than 500 mg/L in a multi-compartment cell. Average power requirements are 1 kWh/m^3 of water per 1000 mg/L of TDS removed for waters with initial TDS values of 10,000 mg/L and less. Since power requirements rise sharply with higher initial values in this method compared to distillation and freezing, this process is adopted only for waters containing less than 10,000 mg/L of dissolved solids.

10.15.6 Pre-treatment Requirement for Membrane-based Treatment

As we have now seen, membranes are fine filters, similar to gravity sand filters, that provide a physical barrier for the removal of turbidity-causing particles and many water-borne pathogens. However, for the smooth and trouble-free running of membranes (specially pressurised ones), the feed water (to membranes) must be properly pre-treated.

One such pretreatment for “Out to In” type of membranes is either a standard sand filter or a 100 to 150 micron (self-cleaning or manual) filter. This ensures that the feed water does not contain more than 5 to 10 mg/L of suspended solids. For the “In to Out” category of membranes, the pretreatment becomes even more important since the inside diameter of membrane is quite small, and can get quickly choked, demanding frequent backwashing and cleaning.

Similar to gravity sand filter choking, membrane filters also get choked (or clogged) after a certain time interval by the accumulation of particles on the surface of the membranes. The deposits need to be removed before membranes are again put into the filter service cycle. This operation of getting away the dirt (or suspended solids) is called backwash or back pulsing. Like in the case of backwashing of sand filters, a small amount of filtered water is typically pushed in the opposite direction (to the filtration cycle). This way, the solids accumulated either on the membrane surface or inside the membranes are flushed out. The membrane backwash process occurs frequently (often every 30 to 40 minutes of operation) for a very short duration (typically two to three minutes).

As membrane filtration systems function, dissolved pollutants in the water may slowly precipitate on the membrane’s surface, lowering the membrane’s permeability and increasing the trans-membrane pressure (TMP) required to filter the water. When the TMP exceeds the normal operating limits for the system, the membranes require a recovery clean, also called a clean-in-place (CIP), to remove the precipitated contaminants and restore the membrane permeability.

10.15.7 Design Guidelines for RO-based System

For designing an RO system, the following information is necessary:

(i) The complete ionic analysis of feed water includes, but not limited to,
   a) the concentration of cations like Ca, Mg, Na, K, Al, Fe, Mn, Boron, etc.;
   b) the concentration of anions like Cl^-, SO_4^{--}, CO_3^{--}, HCO_3^-, NO_3^-, colloidal and reactive silica, etc.;
   c) the presence of organics (BOD/COD/TOC values);
   d) overall TDS;
   e) pH and temperature with all possible variations;
   f) expected recovery (and rejection) of ions;
   g) operating hours.

(ii) The above analysis is to be fed into the “projection software” provided by the selected membrane manufacturer.

(iii) Any amount of excess anion/cation will be shown in “warnings”, i.e., the system may not function properly if the input is not properly adjusted.

(iv) More often than not, projection software also needs inputs regarding the type of water, viz. groundwater/with pretreatment/UF treated, etc.

(v) If all the parameters are within design guidelines by the particular manufacturer, select the number of membranes and hence the number of pressure tubes.

(vi) Projection software also indicates pump pressure required with a lot of other data.

(vii) There are variables available in the design/sizing of the RO system, such as
   a) the use of 4” or 8” membranes (depending on the flow to be handled);
   b) recovery percentage;
   c) feed pH (adjustable with the addition of acid/alkali, etc.);
   d) variations in the number of membranes per pressure tube (usually varying from 2 to 7 membranes per tube) and different “arrays”;
   e) a variety of membranes that offer different performance.

(viii) There is also a provision to insert an “energy recovery device” (ERD) in case the designer is interested in recovering some energy from the rejects stream. The use of such devices is to be done in consultation with OEMs of ERD manufacturers.

10.15.8 Energy Efficiency of RO

To recover freshwater from seawater, RO technology takes a significant amount of energy. Because of advancements in RO technology, seawater RO (SWRO) has become the primary form of large-scale desalination around the world. However, the specific energy consumption (SEC) of SWRO remains significantly greater than that of surface water treatment and indirect potable recycling, making SWRO less cost-effective than other potable water production solutions. Furthermore, when non-renewable energy sources are employed to meet SWRO energy demand, larger quantities of greenhouse gas are emitted than when lower energy alternatives are used.

The energy causes 60% of the total cost in RO systems. Hence, optimisation of energy is essential to keep the overall cost optimum.

Following designs are used for efficient use of energy:

1) Use of ERDs
2) Selection of optimal flux
3) High efficiency of HPP
4) Use of VFDs
5) Use of high-efficiency drives
6) Prevention of biofouling in RO membranes

10.15.9 Membrane distillation

Membrane distillation (MD) is a thermal separation technique that uses a hydrophobic membrane to separate water vapour from water containing dissolved salts. The driving force in the MD process is the difference in vapour pressure between two sides of the membrane induced by vacuum. This process is particularly useful in desalination, treatment of brackish water and industrial wastewater treatment. The water recovery can be > 80% and with solar power, the energy requirement can be brought to a minimum. It can be integrated with existing roof-top solar water heaters to generate the driving force. Hence, it can be used at domestic scale as well. It is less prone to scaling and fouling issues and has low energy consumption as compared to other desalination techniques.

10.16 Desalination

It is recommended to use RO membranes for the treatment of

a) brackish water reverse osmosis (BWRO)
b) seawater desalination (SWRO)

Details of BWRO and SWRO are discussed in following sub-sections.

10.16.1 BWRO Systems

BWRO systems are typically designed with raw water TDS ranging from about 1500 to 10,000 mg/L. Beyond this TDS level, SWRO is required to be employed. For groundwater or surface water having TDS less than 1000 mg/L, RO treatment is not required or not desirable.

RO process itself is extremely versatile but needs the feed water to be treated with great care. Some of the limiting conditions (of feed water to BWRO) are

- Total suspended solids : Below detectable limits (BDL)
- Silting density index (SDI) : Preferably below 3.0, but less than 5.0
- Oxidising chemicals : BDL
- Silica (colloidal) : BDL
- Other heavy metals : BDL
- TDS : Less than 10,000 mg/L
- Iron/manganese : Less than 0.1 mg/L
- Bacterial load : Preferably NIL

To achieve these parameters, in most cases, some pretreatment is necessary. Unit operations and unit processes involved in such treatment depend on the (raw water) source. The following block diagram indicates the combination of possible schemes (Figure 10.14).

BWRO systems usually operate at low or moderate pressure (8 to 20 Bar-G). Feed pressure required (at the membrane feed end) is usually 2 to 4 Bar-G and can have recoveries up to even 90%, depending on the array. If the feed water contains a large amount of heavy metals, silica, boron, etc., more exhaustive pretreatment is called for.

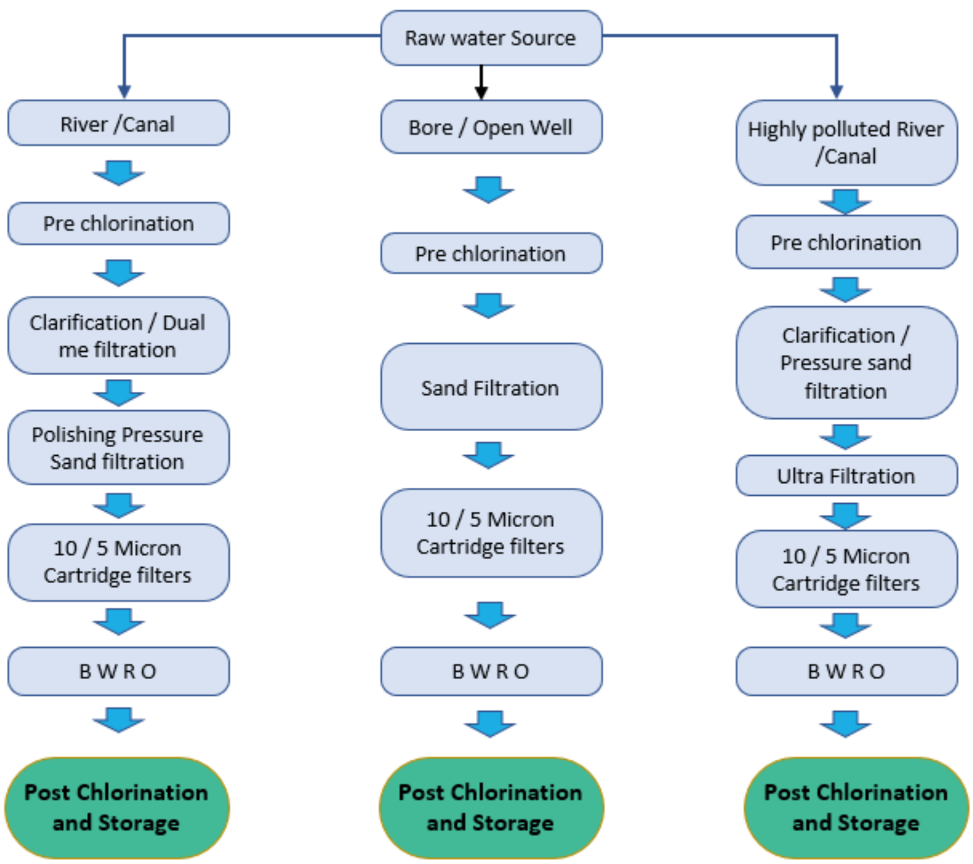


Figure 10.14: Block Diagram showing combination of possible schemes with BWRO system

10.16.2 Seawater Desalination (SWRO)

Pressure-driven RO has become the most preferred method to desalinate seawater across the world over the last two decades. RO is a membrane-based separation process, where high pressure is applied to the concentrated influent water to overcome the osmotic pressure through a semi-permeable membrane, to enable the separation of molecular/dissolved impurities from the water. This splits the feed into reject stream rich in salt molecules and another stream leans in salts thus reducing the TDS of the water.

Limitations of Conventional RO

Typically, SWRO process takes in feed water between 30,000 and 40,000 mg/L TDS and can concentrate reject streams up to 70,000 mg/L TDS. Concentration above this limit is capped by the prohibitive energy requirements. Thus, the ideal permeate recovery from the process is capped at ~ 40%.

Current desalination plants world over, need to increase their intake (by up to 2.5 times) due to this recovery constraint to meet ever-growing permeate demands. Further, brine generated from the SWRO process is sent back to the sea as it cannot be treated further.

10.16.3 Counter-flow Reverse Osmosis (CFRO)

CFRO system has been developed and scaled to take in existing brine streams (> = 70,000 ppm TDS) from your SWRO process to

- recover more water from brine streams and thereby increase recovery; and
- considerably reduce final brine generation.

CFRO is an advanced process innovation based on the conventional RO process developed for brine handling needs and starts where conventional SWRO systems stop. CFRO systems extract high-quality permeate from saline water sources, including high-salinity brine streams, at lower pressure, with less energy and lower capital and operating costs than alternative approaches.

With minimal intervention and maximised permeate extraction enable quick project turn-around, be it brownfield expansion or end-to-end greenfield desalination needs.

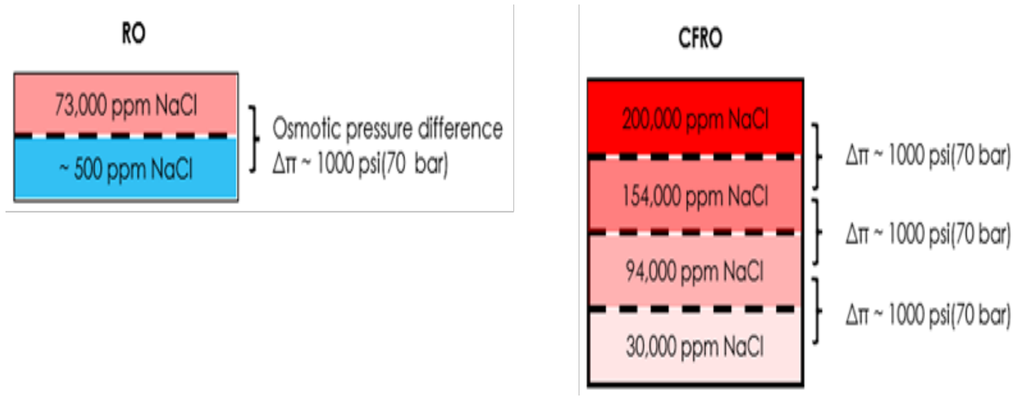


Figure 10.15: CFRO Working

The CFRO process works on the principle of osmotic assistance; diluted brine from succeeding stages is recirculated back into the system to decrease the relative osmotic pressure requirements and thus concentrate reject streams up to 200,000 ppm. This concentration level is comparable to thermal evaporators. Furthermore, the CFRO achieves the same with the standard spiral wound membrane architecture and similar operating pressures as conventional SWRO systems. Figure 10.15 shows CFRO working and Figure 10.16 shows CFRO configuration.

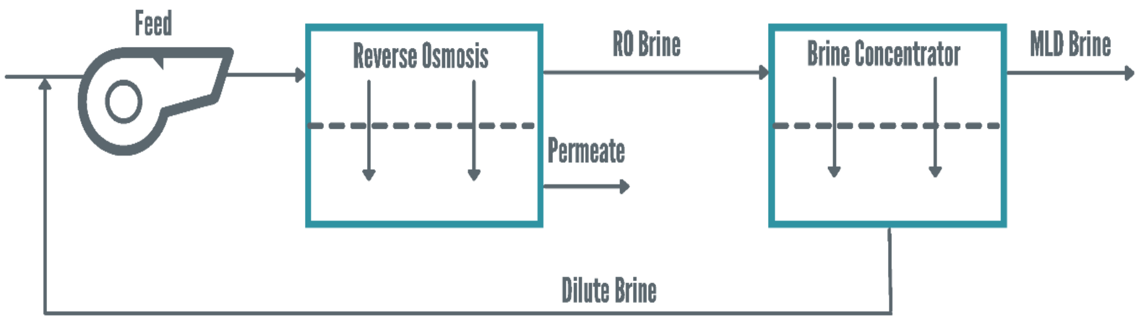


Figure 10.16: CFRO Configuration

Novel advantages for desalination needs

- high recovery desalination solution up to 90%;
- minimise intake/outfall structures required for your desalination plants;
- quick SWRO project turn-around to match your expansion ambitions;
- meet future DM water quality demand with current desalination assets.

10.16.4 Design Criteria of Desalination Plant

The saline waters in India including the seawater, coastal, and inland brackish waters having salinity more than the limits recommended for potable use require treatment through the installation of desalination plants. Typically, water with a TDS concentration higher than 500 mg/L and not higher than 15,000 mg/L is classified as brackish. Seawater is classified as natural water sources with TDS values greater than 15,000 mg/L, such as sea, bay, and ocean waters. This concentration can range up to 36,000 mg/L at various locations and depths along the seacoast.

There is no one “best” method of desalination. Generally, distillation and RO are recommended for seawater desalination, while RO and ED are used for brackish water desalination. However, the selection and use of these processes should be very site-specific, they must be selected very carefully.

One of the major considerations in the selection of a desalination process should be its cost and maintenance. However, despite the substantial costs involved, the availability of desalinated water in arid zones can be a boon to that area. Where the water is salty, alternative water for consumption is often transported over long distances by truck or animal. When the water is sold, its unit price often exceeds that of desalinated water. Therefore, the economic conditions to support desalination already exist in many water-short areas.

Apart from this, the following criteria should be given due consideration for site selection and designing a desalination system of water supply.

Site selection is critical for desalination plant design, finance, building, and operation. The following criteria should be considered when choosing a site:

a) It should be suitably located in a marine environment where an adequate quantity of feed water with a reasonably good, uniform, and steady quality of feed seawater is abstracted at a reasonable cost.
b) The area extent and shape (topography and geometry) must be appropriate so that the marine intake head structures, the marine pipelines, the inland pit, the seawater pumping station, the inland pipelines, the main facility structures, the post-treatment system, the

product delivery sub-system, and the power supply system (independent of national grid substation) are adequately accommodated and optimally located so that civil, electrical, piping interconnections, and other works costs are minimised.

c) It should be at a location where the brine, backwash wastewater, and other wastes are disposed of without environmental adverse effects.
d) Geologically and oceanographically are suitable for the construction and erection of the various structures at reasonable costs.
e) Environmental, town planning, and rural planning regulations, law requirements, and restrictions are met.
f) The desalination plant shall have the social acceptance of the neighbouring communities and other authorities.
g) Must be located in a place where access and interconnections to the power supply grid (or independent power production or alternate source of energy, especially renewable energy resources) and the water supply networks are technically and economically feasible.

10.16.5 Seawater Intake

Site condition
Physical site parameters, as well as meteorological and oceanographic data, must be considered while evaluating a site for seawater intake. In addition, potential sources of contamination such as fouling by marine organisms, oil spills, or other pollution should be evaluated.

When considering the location of an intake structure, it is important to remember that the ocean is in constant motion and is constantly changing. Water levels vary on a daily basis as the tide level changes. In addition, forces caused by waves and currents are constantly at work modifying the shoreline and the profile of the sea floor near the shore.

Water quality
The surf zone is the area in which waves approaching the shoreline break. Breaking waves create a great deal of turbulence. The churning motion of breaking waves suspends particles from the bottom, making the water in the surf zone more turbid and with higher amounts of suspended solids. For this reason, it is not advisable to take seawater directly from the surf zone. As a result, wave forces on structures can be reduced by situating them at an appropriate depth below the water’s surface.

Water temperature also varies with depth. It is important to keep water temperatures within a properly designed range for seawater intake. To reduce temperature fluctuations that can be experienced in stratified surface layers, it is preferred that seawater be obtained from a deeper layer.

One of the first considerations then, concerning the location of an intake for a desalination plant, is the proximity to the shore of a location deep enough to obtain cooler, less turbid water. Either a pipe or channel-type intake can then be selected, depending on seabed conditions, to bring the water to the plant. In alluvial soil formation on the shoreline, seawater may be collected through the construction of beach wells in coastal alluvial aquifers.

Seabed conditions
The seabed conditions will be one of the primary factors in determining the type of seawater intake structure for a particular location. If the intake is to draw water from the open coastline and not from a sheltered lagoon (preferred location, if available and pollution free), the two most common types of intakes are the pipe and the channel intake.

Pipe-type intake

In a pipe-type intake, a pipeline is run from the shoreline out past the surf zone (Figure 10.17 and Figure 10.18). For sandy bottoms, a trench is usually dredged, the pipe is laid, and the trench is backfilled. For bottoms composed primarily of rock or coral, dredging and backfilling would be difficult and expensive. Pipelines are typically buried for anchorage and protection from wave forces and currents. Other methods of anchorage besides trenching and backfilling have been employed, including multi-helix anchors, concrete saddles, engineered backfill, and grouted neoprene-impregnated nylon bags or pillows filled with grout. There are other problems associated with laying pipelines on a rocky bottom. Pipes laid on uneven terrain will only be supported at the high spots. This bridging between support points results in stresses that must be carried by the pipe. In addition, the danger of abrasion of the pipeline is increased with rock bottoms. For these reasons, rock bottoms are not conducive to the installation of a pipe-type intake.

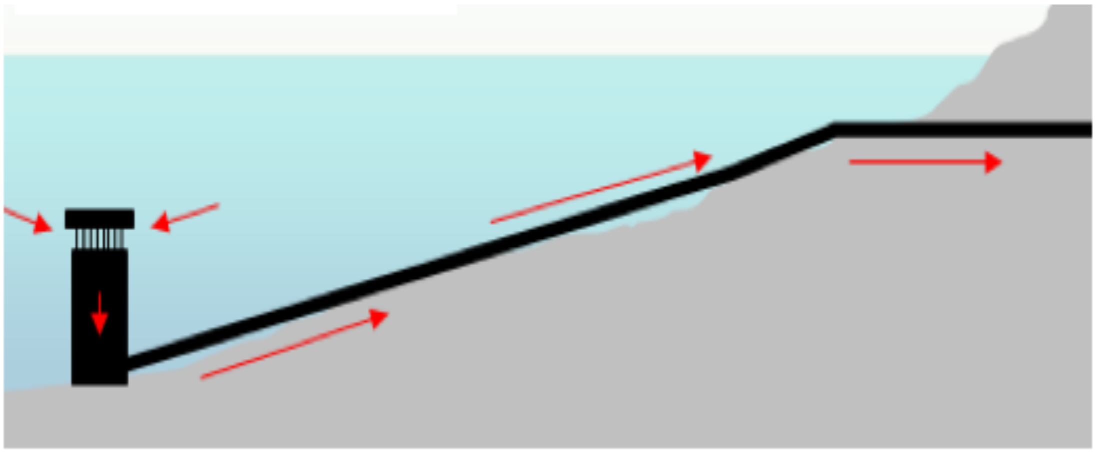


Figure 10.17: Side View of Sub-surface Intake

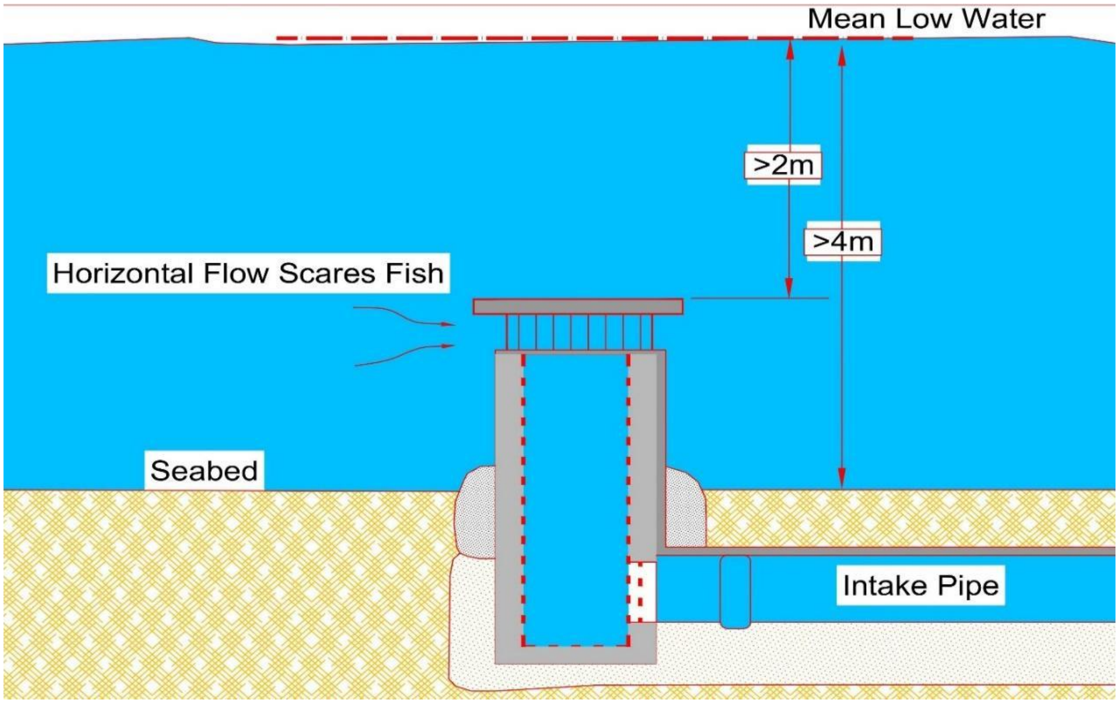


Figure 10.18: Pipe-type Intake

Channel-type intake

A channel-type intake can be constructed either by installing rocks to form walls along each side of the channel or by dredging (Figure 10.19). In sandy bottoms, there is a tendency for the movement of sand due to littoral transport. This littoral transport can either act to erode sand from the channel walls or result in sand being deposited in the channel and could result in the channel requiring re-dredging to keep the channel open. In addition, wave action in the channel would result in sand and silt being placed in suspension causing higher turbidity. For rocky bottoms, the problems of littoral transport, erosion, and suspension of particles due to wave activity are not significant considerations in designing channel-type intakes. The design of a channel-type intake on rock bottoms would primarily be concerned with resisting wave forces and eliminating debris.

As a result, a sandy bottom favours the construction of a pipe-type intake system, whereas a rocky bottom favours the installation of a channel-type intake system.

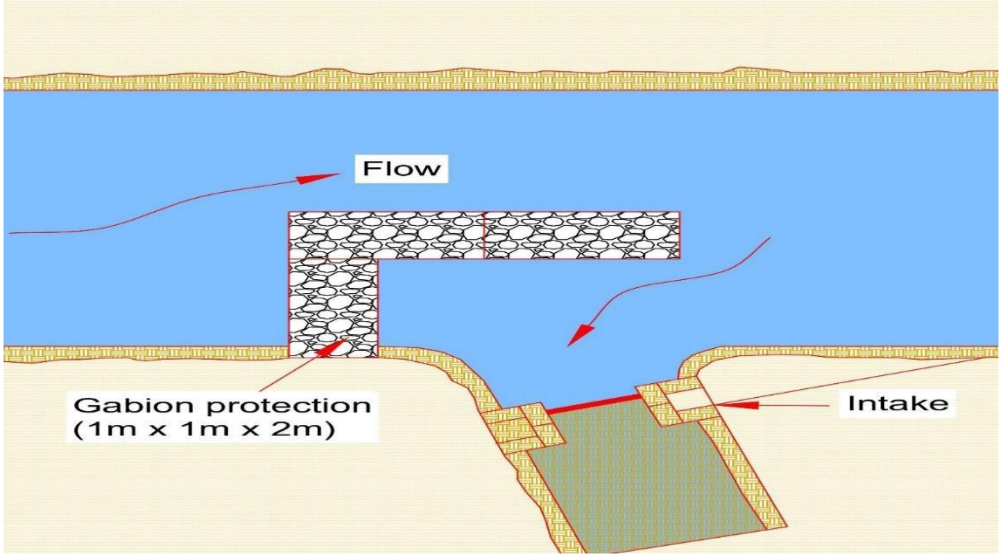


Figure 10.19: Channel-type Intake (Gabion Protection)

Non-surface intakes

Non-surface intakes include beach wells, seabed filtration, and inflow galleries. These types of intakes are variations on the same basic approach of taking filtered seawater from below the surface near the shoreline (Figure 10.20). Each of these intakes has advantages, capabilities, suitability, and cost-effectiveness for different site conditions.

Due to natural filtration and underground detention, beach wells and galleries extracting groundwater of seawater quality can deliver a more consistent quantity and higher quality water than surface intakes. Although experience with similar beach wells and galleries as intakes for SWRO plants is limited, non-surface intakes promise an opportunity for improved efficiency, reliability, cost-effectiveness, and performance of desalination plants, particularly SWRO plants, due to their positive effects on feed-water quality.

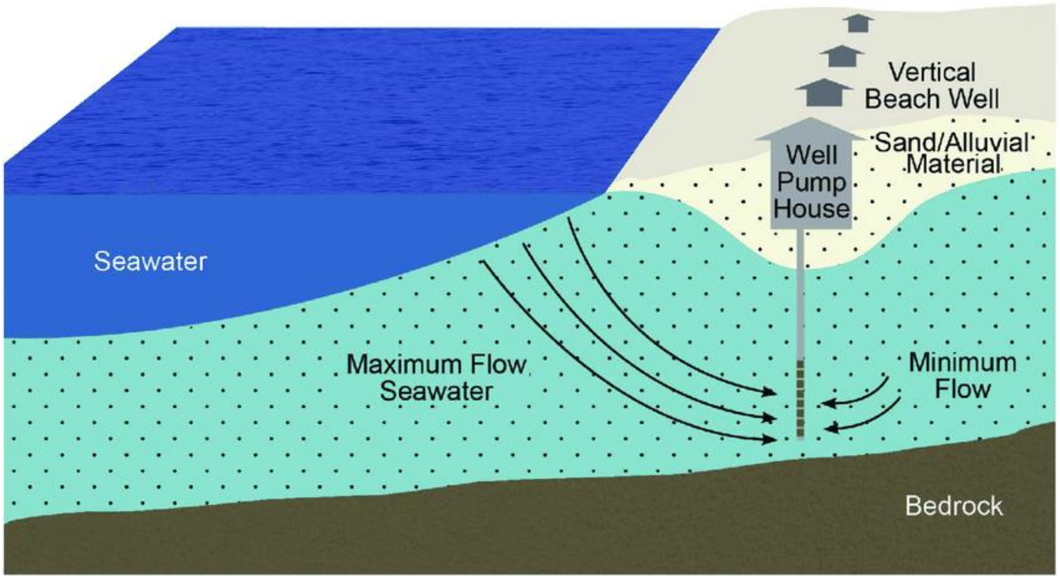


Figure 10.20: Non-surface Intake

Other factors

The oceans are extremely dynamic systems constantly in motion as a result of external forces, such as wind, and internal forces, such as temperature and salinity gradients. The rotation of the Earth results in oceanographic currents and wind forces cause waves and also affect the currents. Seismic forces on the Earth may also result in disruptive waves in the form of tsunamis and seiches (standing waves oscillating in a seawater body).

Environmental impact assessment: A detailed environmental impact assessment of the SWRO project has to be carried out before taking final decision on the site selection and implementation of the project.

10.16.6 Design of Desalination Plant

Table 10.4 provides a general indication of the range of source water salinity for which distillation, RO separation, Electrodialysis (ED), and Ion exchange can be applied cost-effectively for desalination.

Table 10.4: Range of Source Water Salinity

Separation process	Range of source water TDS concentration for cost-effective application, mg/L
Distillation	20,000–100,000
RO separation	2000–46,000
Electrodialysis	200–3000
Ion exchange	1–800

For processes with overlapping salinity ranges, a life-cycle cost analysis for the site-specific conditions of a given desalination project is typically applied to determine the most suitable desalination technology for the project.

10.16.6.1 Thermal Desalination

All thermal desalination plants have five key streams: source water (seawater, brackish water, or brine) used for desalination; steam needed for evaporation of the source water; cooling water to condense the freshwater vapour generated from the source water's evaporation; low-salinity distilled water (distillate); and concentrate (brine), which contains the salts and other impurities separated from the source water.

Multi-stage flash distillation (MSF), multi-effect distillation (MED), and vapour compression (VC) are the three most popular types of thermal desalination technologies. Each of these classes of technology has evolved over the past 40 to 60 years toward improvements in efficiency and productivity. The three types of thermal technologies mainly differ by the temperature and pressure at which the source water is boiled to generate freshwater vapour.

a) Multi-stage flash distillation (MSF)

In the MSF evaporator vessels (also referred to as flash stages or effects), the high-salinity source water is heated to a temperature of 90 to 115 °C (194 to 239 °F) in a vessel (the heating section) to create water vapour. The pressure in the first stage is maintained slightly below the saturation vapour pressure of the water. So, when the high-pressure vapour created in the heating section enters the first stage, its pressure is reduced to a level at which the vapour “flashes” into steam. Steam (waste heat) for the heating section is provided by the power plant co-located with the desalination plant. Each flash stage (effect) has a condenser to turn the steam into the distillate. The condensers are equipped with heat exchanger tubes, which are cooled by the source water that is fed to the condensers.

Entrainment separators (mist eliminators or demister pads) remove the high-salinity mist from the low-salinity rising steam. This steam condenses into pure water (distillate) on the heat exchanger tubes and is collected in distillate trays, from where it is conveyed to a product water tank. Distillate flows from stage to stage and is collected at the last stage. The concentrate (brine) is generated in each stage and after collection at the last stage, some of it typically is recycled to the source water stream to reduce the total volume of source water that must be collected by the intake for desalination. The recirculated brine flowing through the interior of the condenser tubes also removes the latent heat of condensation. As a result, the recirculated brine is also preheated close to maximum operating temperature, thereby recovering the energy of the condensing vapour and reducing the overall heating needs of the source water. This “brine recycle” feature has been adopted in practically all of the most recent MSF facility designs and allows significant improvement of the overall cost competitiveness of MSF installations.

Each flash stage typically produces approximately 1% of the total volume of the desalination plant’s condensate. Due to the fact that a typical MSF unit contains 19 to 28 effects, the overall MSF plant recovery (i.e., the volume of distillate expressed as a percentage of the total volume of processed source water) is normally 19 to 28%. For comparison, RO seawater desalination plants have a recovery of 40 to 45%. The latest MSF technology has 45-stage units, i.e., can operate at 45% recovery. This feature allows it to compete with RO systems in terms of recovery.

Historically, MSF was the first commercially available thermal desalination technology applied to the production of potable water on a large scale, which explains its popularity. Over 80% of thermally desalinated water today is produced in MSF plants.

b) Multiple-effect distillation (MED)

In MED systems, the saline source water is typically not heated; cold source water is sprayed via nozzles or perforated plates over bundles of heat exchanger tubes. This sprayed feed water boils, and the resulting vapour flows through mist eliminators, which remove brine particles from the vapours. The feed water that turned into vapour in the first stage (effect) is introduced into the heat exchanger tubes of the next effect. Because the next effect is maintained at slightly lower pressure, although the vapour is slightly cooler, it still condenses into freshwater at this lower temperature. This process of reducing the ambient pressure in each successive stage allows the feed water to undergo multiple successive boiling without the introduction of new heat. Steam flows through the exchanger tubes is condensed into pure water and collected from each effect. Heating steam (or vapour) introduced in the heat exchanger tubes of the first effect is provided from an outside source by a steam ejector.

The MED system is also equipped with a brine recycle system, which allows the introduction of warmer-than-ambient water in the first effects of the system, thereby reducing both the volume of feed water that must be collected by the plant intake system and the overall energy needs of the system.

The main difference between the MED and MSF processes is that while vapour is created in an MSF system through flashing, evaporation of feed water in MED is achieved through heat transfer from the steam in the condenser tubes into the source water sprayed onto these tubes. This heat transfer at the same time results in condensation of the vapour to freshwater. MED desalination systems typically operate at lower temperatures than MSF plants (maximum brine concentrate temperature of 62 to 75 °C versus 115 °C).

c) Vapour compression (VC)

The heat source for VC systems is compressed vapour produced by a mechanical compressor or a steam jet ejector rather than a direct exchange of heat from steam.

In VC systems, the source water is evaporated and the vapour is conveyed to a compressor. The vapour is then compressed to increase its temperature to a point adequate to evaporate the source water sprayed over tube bundles through which the vapour is conveyed. As the compressed vapour exchanges its heat with the new source water being sprayed on the evaporation tubes, it is condensed into pure water. A feed water preheater (plate-type heat exchanger) is used to start the process and reach evaporation temperature.

VC and MED work based on similar principles. However, while in MED, the steam produced by source water evaporation is introduced and condensed in a separate condenser located in the downstream effect; in VC, the steam generated from the evaporation of new source water is sprayed on the outside surface of the heat exchanger tubes, recirculated by the vapour compressor, and then introduced into the inner side of the same heat exchanger tubes, where it condenses to form distillate.

10.16.6.2 Membrane Desalination

Two general types of technologies currently used for membrane desalination are RO and ED which are described in earlier sections. Table 10.5 provides a comparison of the removal efficiencies of distillation, ED, and RO systems for key source water quality compounds. One important observation from this table is that, as compared to distillation and RO separation, ED desalination only partially removes nutrients from the source water. This explains why EDR is frequently regarded as more desirable than RO or thermal desalination.

Table 10.5: Contaminant Removal by Alternative Desalination Technologies

Contaminant	Distillation (%)	ED/EDR (%)	RO (%)
TDS	>91.9	50–90	90–91.5
Pesticides, organics/VOCs	50–90	<5	5–50
Pathogens	>99	<5	>910.99
TOC	>95	<20	95–98
Radiological	>99	50–90	90–99
Nitrate	>99	60–69	90–94
Calcium	>99	45–50	95–97

Magnesium	>99	55–62	95–97
Bicarbonate	>99	45–47	95–97
Potassium	>99	55–58	90–92

One important observation from this table is that, as compared to distillation and RO separation, ED desalination only partially removes nutrients from the source water. This fact explains why EDR is often considered more attractive than RO or thermal desalination (which removes practically all minerals from the source water) if the planned use of the desalinated water is for agricultural purposes, i.e., generating fresh or reclaimed water for irrigation of crops.

Construction and equipment costs for BWRO and EDR systems of the same freshwater production capacity are usually comparable, or EDR is less costly, depending on the RO membrane fouling capacity of the source water. However, since the amount of electricity consumed by EDR systems is directly proportional to the source water's salinity, at salinities of 2000 to 3000 mg/L, the energy use of EDR systems usually exceeds that of BWRO or NF systems for source water. As a result, EDR systems are not as widely employed as RO systems for BWRO desalination and are never used for SWRO. It should be pointed out, however, that salinity is not the only criterion for evaluating the cost-competitiveness of EDR and BWRO systems. Often, other compounds such as silica play a key role in the decision-making process.

Over the past two decades, RO membrane separation has evolved more rapidly than any other desalination technology, mainly because of its competitive energy consumption and water production costs. The all-inclusive energy consumption for freshwater production of thermal desalination plants is typically much higher than that for brackish or seawater desalination. BWRO desalination yields the lowest overall production costs of all the desalination technologies. It is also worth noting that the most recent MED projects undertaken in recent years have been finished at costs equivalent to those of comparable sized SWRO facilities. For the majority of medium and large projects, however, SWRO desalination usually is more cost-competitive than thermal desalination technologies.

10.16.7 Brine Management

Brine (also known as membrane concentrate, reject brine, and wastewater) is a high-concentration salt (NaCl) solution in water (H_2O). Brine can refer to salt solutions with concentrations ranging from around 3.5% to roughly 26%. Brine is formed naturally from evaporation of ground saline water, but it is also produced during sodium chloride mining. It is also a by-product of many industrial processes, such as desalination, so it requires wastewater treatment for proper disposal or further utilisation (fresh water recovery).

Disposal of Brine

Disposal of brine in desalination plants is of much significance both from economic and environmental standpoints. Improper surface disposal has the potential for polluting the groundwater resources that are used as feed water for many of the desalination plants. The groundwater pollution is likely to result from high salinity and the presence of other harmful chemicals in the brine. For the disposal of rejected brine from inland desalination plants, the following solutions should be examined:

a) Pumping into specially designed lined evaporation ponds;
b) Deep well injection;
c) Disposal into surface water bodies;
d) Disposal through pipelines to municipal sewers;
e) Concentration into solid salts; and
f) Irrigation of plants tolerant to high salinity (halophytes).

Options a) and f) are seasonal and, therefore, a backup alternative is required to improve their reliability.

The following factors influence the selection of a disposal method:

a) The volume or quantity of concentrate;
b) The quality or constituents of concentrate;
c) The physical or geographical location of the discharge point of the concentrate;
d) The permissibility of the option;
e) Public acceptance;
f) Capital and operating costs; and
g) The facility's ability to be expanded.

The main environmental issues for an appropriate location for brine discharge are:

a) Finding a region with no endangered species or stressed aquatic ecosystems;
b) Finding a location with a high underwater current to allow fast dissipation of high-salinity discharge;
c) Avoiding areas with ship traffic to avoid damaging the brine discharge system and altering the mixing pattern;
d) Identifying a discharge point near the shoreline or relatively shallow water to minimise construction;
e) Ensuring that discharged brine descends to the seabed and travels along the seabed thereafter;
f) Ensuring no dressing or alteration of the sand dunes, or natural features including landscape changes for beautification, recreation, and other such purposes; and
g) To meet the CPCB's water quality standards for coastal waters marine outfalls (Vide: EPA, 1986 -GSR 7, dated December 22, 1998).

Importance of Brine Reduction

Brine salts change the chemical and physical properties of soils. Brine has a deleterious impact on soils in a variety of ways due to the high concentrations of soluble salts [most notably sodium chloride ($NaCl$)]. Many biological species are poisoned by chloride levels in and around the spill location. Sodium is a natural dispersant, causing soils to inflate and spread. The salts in brine reduce the plant's ability to absorb water and nutrients. High salt concentrations in the soil limit the plant's ability to absorb water even when ample water is available in the soil, causing the plant to exhibit drought symptoms. This is due to an osmotic action, which causes water to migrate from low salt concentration locations, such as the roots, to high salt concentration areas, such as the soil.

Plant growth is reduced in damaged areas due to the effects of excessive salt concentrations on soil and vegetation. This is exacerbated by the incapacity of many seeds to germinate.

Methods of Brine Reduction

The following methods are generally used for reduction of brine:

(i) Membrane treatment system;
(ii) Thermal evaporation;
(iii) Evaporation ponds.

(i) Membrane treatment system

RO is the membrane system most widely used to desalt brine waters. RO generates freshwater as well as a more concentrated brine, which is referred to as RO brine, reject, or concentrate. This brine concentrate will typically include dissolved salts and chemical concentrations that are close to scaling limitations. If you intend to use a thermal system to further concentrate the brine or generate solids, you must treat it to reduce the scaling potential.

(ii) Thermal evaporation
When considering thermal evaporative systems, optimising freshwater recovery through lower cost membrane systems before adopting expensive thermal systems would result in the best project economics. Based on their residual outputs, thermal systems are classified into two types: (1) evaporators, which create concentrated, low volume brine but do not precipitate solids, and (2) crystallisers, which exceed salt saturation and produce solids.

(iii) Evaporation ponds
Evaporation ponds are an artificial solution to waste brine discharge from inland surface water. The water evaporates under the correct climatic circumstances, allowing you to discharge more brine into the ponds. One limitation of ponds is that they require large areas of land to increase the surface area where the water can evaporate, and can represent a future environmental liability due to either animal entry or future decommissioning. If solids must be recovered for disposal or reuse, numerous evaporation ponds may be required to alternate between brine evaporation and solids extraction. Evaporation is also faster in hotter, arid areas.

10.16.8 Capacitive Deionization (CDI)

CDI is an emerging technology that can be used to treat brackish water. This principle of this technique is electro adsorption of ions on the surface of the electrodes. Various electrode materials used for CDI are activated carbon, carbon cloth, ordered mesoporous carbon, carbon nanofibers, carbon nanotubes/multiwall carbon nanotubes (CNTs/ MWCNTs), and graphene and graphene-based composites. A CDI cell comprises of a pair of porous electrodes separated by a non-conducting separator. When a potential difference (below 2V) is applied to the electrodes, they get charged and the ions present in the feed water migrate to the electrical double layer (EDL) of the oppositely charged porous electrodes. The ions are electrostatically held at the EDL until an equilibrium is reached when no more ions get adsorbed. Desorption happens when the potential is reversed, or the external power supply is shorted and the ions leave the electrodes as the brine stream thereby regenerating the electrodes. Energy can be recovered from the charge that is leaving the cell. Some of the important parameters for defining the performance of a CDI unit are maximum salt adsorption capacity, average salt adsorption rate, charge efficiency and charge storage capacity. CDI has several advantages over other desalination techniques such as energy efficiency (high pressure pumps are not required), less water rejection compared to RO, possibility to work on solar/wind power as it requires only 0.8-2.0 V, and possibility of energy recovery as it works as a capacitor. The limitation of CDI lies in the availability of electrodes with high electro adsorption capacity.

In the last decade, there has been many developments in membrane capacitive deionization (MCDI) wherein, an ion exchange membrane is introduced on top of the porous electrodes. This enables complete regeneration of the electrode surface during the desorption step. Flow electrode/flow electrode capacitive deionization (FCDI) cell is yet another advancement wherein a carbon slurry is continuously passed between the electrode compartments and the ions get adsorbed. The desorption step takes place in a separate compartment downstream. The continuous flow of uncharged carbon particles increases the desalination capacity by increasing the effective capacitance. Therefore, FCDI can desalinate water of higher salinity and even sea water having TDS of 31g/L with an efficiency of 95% and can overcome the limitation of conventional CDI.

Currently CDI has been has implemented for brackish water desalination and a water recovery of up to 82% has been achieved, with output TDS below 150 ppm. It can also address ionic contaminants such as fluoride, nitrate, uranium, arsenic, etc., as well. Units with remote monitoring and powered by photovoltaics have also been implemented. Individual units delivering 3-10 KLD are generally preferred.

10.17 Case Studies on SWRO Applications

Such plants, which are under operation in India, are discussed in the section below

Nemmeli, Chennai Metro, Tamil Nadu

The SWRO plant of Chennai Metro at Nemmeli receives the raw seawater from the Bay of Bengal located on the eastern side of India. The output capacity of SWRO is 100 to 110 MLD. Presently, the plant operates at 70 to 75 MLD capacity. As reported, the produced water cost is Rs. 32 per cum. The treated water is supplied (pumped) to the drinking water supply and distribution network of South Chennai at a heavily subsidised rate. The plant was designed and supplied by M/s Wabag-IDE and has been maintained by Wabag since 2013. Seawater has TDS in the range of 36,000 to 39,000 mg/L. It is designed for a maximum TDS value of 41,700 mg/L. The TSS in the fair-weather season is 20 mg/L. However, during monsoon, it goes as high as 300 mg/L. It also receives a moderate load of organics and seashells.

To produce 100 MLD of treated water, 240 MLD of raw seawater is required to be pumped into the pretreatment system. The plant has a deep-sea submerged intake (15 m below water level). Raw seawater is conveyed by HDPE pipe buried under the seabed to the plant's receiving well. The receiving well is provided with vertical travelling bar screens. Most of the seashells and other coarse impurities are arrested and removed by the screens. The raw water is then pumped from receiving well to the pretreatment plant consisting of flash mixers, flocculators, and lamella clarifiers. Pre-chlorination is done at the inlet chamber. Ferric chloride ($FeCl_3$) is dosed as a coagulant. The TSS of treated clarified water ranges from 10-15 mg/L. There are five parallel streams of lamella clarifiers preceded by flocculators. Lamella plates are made out of polypropylene (PP). The clarified water is stored in an underground reservoir.

The clarified water is further pumped at 18 bar (183.55 m H_2O) pressure to Disk Filters followed by UF membranes. The disk filters are self-cleansing and automated to have a cleaning cycle. There are twenty-two parallel chains of skids each comprised of four disk filter vessels. The disk filters produce water of quality less than 10 NTU. It is further conveyed to UF membrane skids (Figure 10.21). The membrane flux has a design value of 85 L/sqm/hr. The treated water from UF has a turbidity of less than 1 NTU. The feed water quality to RO (TSS, turbidity, and organics) is practically BDL due to the combination of Chlorine dosing and polishing off of all suspended particles by disc filters and UF.

The RO feed water is then injected with sodium-meta-bisulphite for de-chlorination of residual chlorine, since RO membranes are generally non-tolerant to chlorine presence. The SDI of RO feed water is achieved in the range of 2 to 3. High-pressure pumps (72 bar) convey the water to RO skids (Figure 10.22). The total recovery rate is 40%. The RO treated water TDS is reduced to a range of 400 to 500 mg/L. The membranes are designed for flux values of 14 to 15 $L/m^2/h$. After the commissioning of the plant, the CIP cycle was 60 days. After seven years of operation, it is 20 days.

For re-mineralisation, part of RO treated water (30%, side stream) is injected with carbon dioxide and then passes through columns of lime flakes. This raises the finished water pH above 7.5. It is further dosed with chlorine (post-chlorination) for complete disinfection. The treated water is stored in the ground reservoir and then pumped to the consumer end (drinking water). The waste, UF, and RO reject (TDS 70,000 mg/L) is disposed of in the sea. An elaborate diffuser arrangement is made at the disposal point so as not to disturb the marine life and environment due to shock loading.

The treated water is of excellent quality, according to the drinking water criteria mentioned in IS 10500:2012. The plant is well designed including pipe and cable routing. It has a sound layout from a process, operation and maintenance logistics point of view. The plant is automated, and manpower is trained and skilled.

During the initial construction of the project in the year 2010, pretreatment consisting of lamella clarifiers was not incorporated. However, to tackle the seasonal variations of TSS, it was required to be provided at a substantial additional cost later. As confirmed during the site visit and with the plant officials, pretreatment of seawater is of utmost importance for plant performance. The degree of pretreatment differs according to the quality of seawater. Seawater quality directly influences the overall project cost.

Figure 10.21: UF Membranes Skids

Figure 10.22: RO Membranes Skids

(**Source:** *Chennai Metropolitan Water Supply and Sewerage Board*.)

10.18 Horizontal or Roughening Filters

The water quality of contaminated surface water can be significantly improved in terms of removing/reducing turbidity when filtered through gravel and sand layers. Therefore, favourable hydrogeological conditions allow polluted and turbid river water to be drawn as clear and safe groundwater from a shallow well located next to a river. These are useful for very small communities where alternative full-fledged treatment is not possible. Roughing filters are typically made up of different-sized filter material that decreases in size in the direction of flow (Figure 10.23 and Figure 10.24). The bulk of the solids is separated by the coarse filter medium located next to the filter inlet. The subsequent medium and fine filter media further reduce the suspended solids concentration. A roughing filter's filter media is made of moderately coarse (rough) material ranging in size from around 25 to 4 mm. Filtration velocity, synonymous with hydraulic load, usually ranges between 0.3 and 1.5 m/h. The accumulated solid matter is periodically flushed out of roughing filters by hydraulic filter cleaning. These filters need to be cleaned manually by excavating the filter material from the filter compartment, and washing and refilling it into the filter boxes. The approximate rate of filtration is given below in a study based in Maharashtra, as shown in Table 10.6 below.

Table 10.6: Approximate rate of filtration is given below from a study based in Maharashtra

Media size	No. of media boxes	Rate of filtration (m^3/m^2)

12–20 mm	5	3
20–25 mm	5	4
25–40 mm	5	5
40–50 mm	5–7	7

Figure 10.23: Horizontal Flow Filters

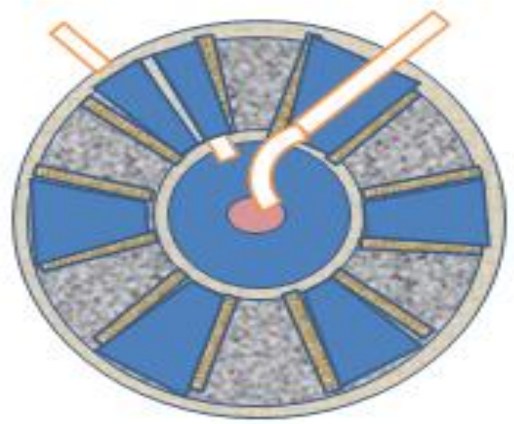


Figure 10.24: Well in Well-type Horizontal Flow Filters

The maintenance can be done by unskilled labour. It is essential to clean the filters before monsoon and after monsoon when they are used as the front pre-filter unit ahead of intake well. For the remaining period, it can be as required based on the observations, normally after 2 to 3 months. It can take any turbidity up to 2000 ppm.

10.19 Water Treatment Technologies for Different Climate

The problems of water treatment in extreme and different climates, high altitudes, and difficult terrains are similar to many other engineering problems in those areas. The major problem is the extreme temperature, which varies between (+) 50 °C to (-) 40 °C. Problems with transportation of equipment, construction, and pipe materials are also present. Behaviour of materials becomes different in extreme climatic conditions. Materials tend to become brittle in extreme cold. The problem of extreme cold combined with lack of oxygen due to height exists in the Himalayas. There is a large variation in atmospheric pressure, which is very low at high altitudes.

10.19.1 Effect of Low Temperature

a) **Physical:** At 4 °C water is at its maximum density. Further reduction in temperature will result in the expansion of the sample of water and at 0 °C, it will solidify and turn to ice having expanded approximately one-twelfth of its volume. This is the reason why bursts in pipes take place by the expansion of water as it changes to ice. The viscosity of water increases with falling temperature. Waters at low temperatures are capable of carrying a greater load of suspended materials than is carried in tropical regions. Settling velocities decrease with temperature. It takes almost twice as much time for particles to settle at 4 °C as it does for them to settle at 23.3 °C, which is very important in the design of sedimentation tanks.
b) **Chemical:** In general, most chemicals react much slower at a temperature near freezing than they do at normal temperatures. Consequently, a longer reaction time is required for satisfactory performance in the treatment units. Coagulation, precipitation, oxidation, water softening reactions, solubility, etc., all are affected due to low temperature.
   The chlorination process also poses difficulty at low temperatures. At temperatures between 1.1 °C and 10.55 °C, solid chlorine hydrate is formed due to removal of the chlorine from the

solution. There is practically no chlorine at 0 °C. Gaseous chlorine cylinders are heated up for chlorine apparatus to work properly.

10.19.2 Effect of High Altitude

Low barometric pressure: Low barometric pressure limits the pump suction head. To summarise, the effect of low temperature is to retard biological and chemical reactions and make the physical state of fluids, solids, and other materials appreciably different. Because low temperatures change most of the warm weather characteristics of water, the conventional methods of water collection, treatment, and distribution need modification for use in low-temperature regions.

10.19.3 Cold Deserts

Cold deserts are primarily located in high altitudes in the Himalayan region of Arunachal Pradesh, Ladakh, Himachal Pradesh, Uttarakhand, etc. The primary source of water in these areas is glacial melt, which is being impacted because of climate change. Further, the water gets frozen in the pipelines during winters disrupting the water supply. Solutions to improve and store run-off water in small tanks — a conventional water harvesting structure, i.e., zing may be investigated. Furthermore, artificial glacial reservoirs can be produced by redirecting run-off and allowing it to freeze and store as a glacier. During early spring, it will serve as both drinking water and irrigation source. Promoting the use of micro-irrigation can reduce the irrigation requirement and increase the drinking water security.

10.19.4 Hilly Areas

In hilly areas, especially at higher altitudes, it is uneconomical to pump water from the valley for a very small number of houses. In these places, adopting spring-based sources, rainwater harvesting, and standalone bore-well systems (if feasible) will be economical. Spring-based systems would require careful identification and delineation of spring sheds, locating the aquifers contributing water to springs, and injection recharging them for sustaining them. Communities may be encouraged to adopt the traditional wisdom of rainwater harvesting (like bamboo-based ones in the NE States) for water security. In such areas, there is a need to focus on Water Quality Monitoring and Surveillance (WQM&S).

10.19.5 Coastal Areas

In coastal areas, augmentation of services can be done with energy-efficient small desalination plants with a high recovery ratio. Further, to avoid the ingress of sea water, sub-surface dykes can be constructed in rivers that can also improve the groundwater-based drinking water sources in the adjoining areas.

10.20 Emerging Contaminants (ECs)

Chemicals that had not before been detected in the water supply are occasionally discovered. These chemicals are known as “emerging contaminants”. ECs are significant because the dangers they pose to human health and the environment are not completely understood. Because these contaminants were introduced or detected relatively recently, there is a knowledge gap in their fate, behaviours, and impacts, as well as treatment strategies for their efficient removal. Furthermore, despite the advances in treatment technologies, the design of existing treatment plants is not suited to remove these ECs. They may be industrial in origin or may originate from municipal (domestic), agricultural, hospital, or laboratory wastewater.

Pharmaceuticals and personal care products (PPCPs) are increasingly being detected at low levels in surface water, and these compounds may be present inadvertently in various compartments of the aquatic environment (e.g., water, sediments, and biota) at concentrations capable of causing detrimental effects to aquatic organisms and the inherent ability to induce physiological effects in humans at low doses. This has become a major concern due to the widespread and rising use of PPCPs in human and veterinary medicine, resulting in their ongoing release into the environment. The substances in question come from three broad categories: pharmaceuticals (PHACs), personal care products (PCPs), and endocrine-disrupting compounds (EDCs). Endocrine disruptors are substances that can interfere with hormone function in the body. Trace quantities of these pollutants have been found in water in several nations. However, they are not limited to the above and may include nanomaterials (NMs), EC metabolites, illegal medications, modified genes, and so forth. PHACs are a class of emerging environmental pollutants that are increasingly being used in human and veterinary medicine. Antibiotics, legal, and illegal pharmaceuticals, and other substances of environmental importance are among them.

Municipal wastewater is regarded as one of the primary discharge sources for developing contaminants, such as non-point and point sources, industry and storms, home wastewater, and water treatment facilities, into the environment. Also, there is a growing concern about sludge management due to the high levels of ECs in them. The current design of WWTPs cannot limit the elimination of developing pollutants and their metabolites where they are released as sewage effluents into rivers or streams with high biodiversity. So far, considerable work has been done regarding the performance of wastewater technologies in the case of nutrient removal, while there is an absence of data on the ability to remove ECs, and additionally on the adverse eco-toxicological impacts of these compounds on surface water bodies.

Per- and polyfluorinated alkyl substances (PFAS) are a class of nearly 4000 man-made chemicals that have lately emerged as emerging pollutants with substantial deleterious effects on human health, even at low concentrations in the parts per trillion range. Non-stick cookware, upholstered furniture, clothing, food packaging, and firefighting foam used to extinguish petroleum fires are all products that contain PFAS. These compounds do not occur naturally in nature. They do not degrade readily and are exceedingly persistent in the environment, particularly in water, as well as in the human body. PFAS has been linked to health risks including developmental effects in foetuses and infants, various forms of cancer, and decreased liver, thyroid, and immune system function.

Thus, the presence of developing pollutants at trace levels in used waters, as well as their behaviour during wastewater treatment and drinking water production, are critical topics that need to be investigated further.

Activated carbon filters are used to reduce PPCPs but cannot completely remove them. Treatments such as RO, NF, and ozone as part of advanced oxidation also reduce PPCPs and other ECs in drinking water. These technologies are already described in earlier sections.

CHAPTER 11: PIPES AND PIPE APPURTENANCES

11.1 General

Pipes and pipe appurtenances are an integral part of a piped water supply system, which are to be selected based on their physical, chemical, hydraulic, structural characteristics and environmental effects (freezing in cold regions, dessert, etc.), possible wear and tear depending on the topography and local conditions (change in the landscape such as in hilly region) on them as well as water characteristics that are subjected to be handled at various stages in the system. Apart from this, the availability of pipes and pipe appurtenances of a particular material at a reasonable economic cost plays a role in selection in various regions.

Piping materials play a vital role in the engineering of the water supply system. The water distribution and transmission system set up to deliver potable water to the consumers' end accounts for an appreciable part of the capital outlay of a water supply system. The success of a project and the cost, to a great extent, depends on piping materials. Hence, proper selection of pipe materials plays an important role in the project economy, and it is always preferred to choose piping materials that meet all technical requirements but are cheaper. There is a lot of variety of piping materials present in the market. Piping material is a broad term and is not limited to only the material of the pipe. It signifies the material of all piping components, pipes, fittings, valves, and other items.

While selecting appropriate pipe material for water supply schemes, it shall be ensured that the specifications for the pipes and other appurtenances should conform to relevant BIS standards as well as the guidelines given in the manual on water supply and treatment, and they must be scrupulously followed while selecting the pipes and appurtenances for the water supply systems. Any pipe (like MDPE pipe) and specials which don't have BIS specification but used in Water Supply System, shall approach BIS for standardization.

Also, the major criterion in pipe wall selection may not only be the temperature and pressure but also the availability of fittings and flanges. Piping is a system, and other items must be considered during selection ensuring their compatibility.

11.1.1 Pipe Materials

Pipelines are major investments in water supply projects, and as such, "Pipes" represent a large proportion of the capital invested in water supply undertakings and are of particular importance. Therefore, pipe materials shall have to be judiciously selected not only from the point of view of durability, life, and overall cost, which includes, besides the pipe cost, the installation and recurring operation, repair, and maintenance costs necessary to ensure the required function and performance of the pipeline throughout its designed lifetime. Every water engineer/designer, while making the choice of pipe material to be used, should give due consideration to the total cost of the pipes, including their transportation and installation costs, their capability to transmit the desired water quantity, and their chemical effect(s), if any, on the water and vice versa the effect of water on the pipe material(s).

11.1.2 Classification of Pipe Materials

Classification Based on Material of Construction

The various types of pipes used are:

I. **Metallic pipes**: Cast Iron (CI), Ductile Iron (DI), Mild Steel (MS), Stainless Steel/Steel, and Galvanised Iron (GI) are sub-classified based on the lining:

A. Unlined Metallic pipes

B. Lined Metallic pipes with cement mortar or epoxy lining

II. **Non-Metallic pipes:** Reinforced Concrete (R.C.), Prestressed Concrete; Cylinder or non-cylinder (PSC), Bar Wrapped Steel Cylinder, Plastic Pipes: PVC (PVC-O, PVC-U), Polyethylene (PE), Glass Reinforced Plastic (GRP)/Fibre Reinforced Plastic (FRP), Asbestos Cement Pressure pipes, etc.

A broad classification of pipes used in water supply systems based on the material of construction is given in Figure 11.1 below:

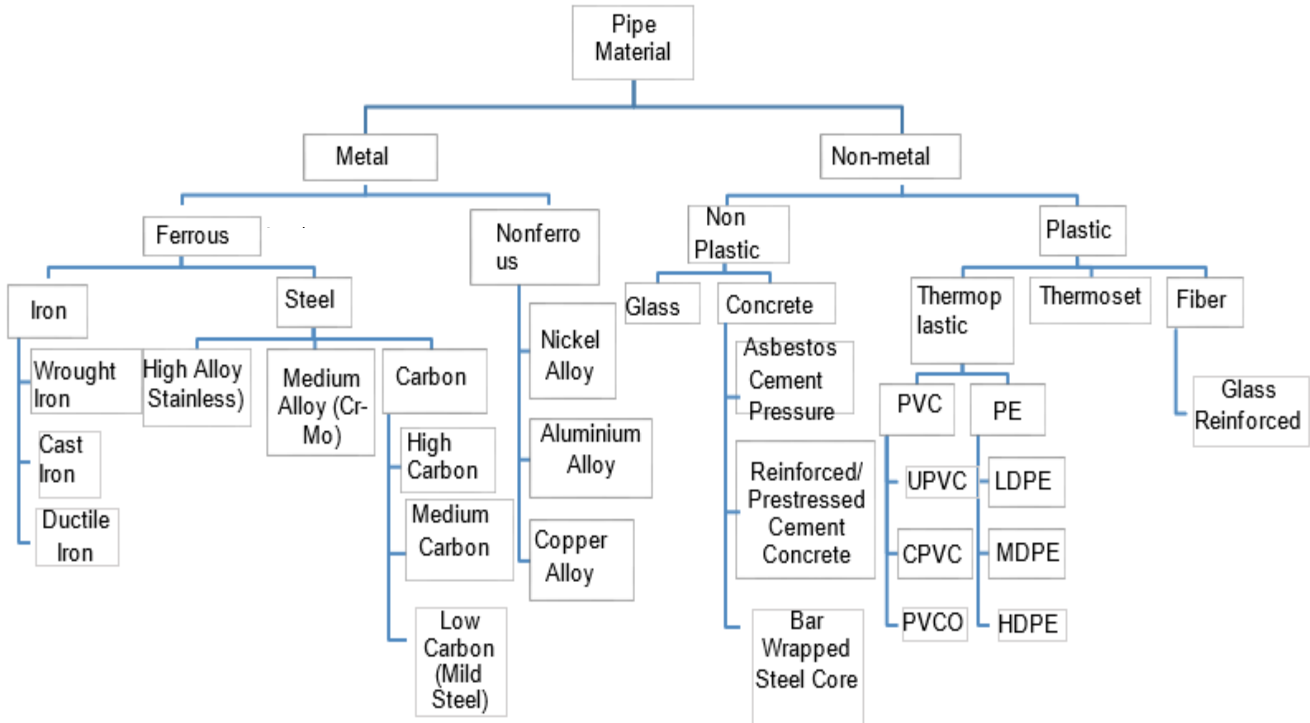


Figure 11.1: Pipes used in water sector

11.1.2.1 Classification Based on Structural Flexibility

Structurally, the pipes must overcome the following forces:

1. Temperature-induced expansion and contraction
2. External loads in the forms of traffic, backfill, and their own weight between supports
3. Unbalanced pressures at bends, contractions, and closures
4. Water hammer
5. Internal pressures equal to the full head of water

Also, pipe-soil interaction is important for the sound structural design of buried pipelines.

Pipes can be grouped as rigid, flexible, and semi-rigid based on their structural flexibility. Flexible pipes such as HDPE, PVC, etc., are defined by their ability to yield under loading without fracturing. Rigid pipes such as concrete and clay, are limited in their ability to yield under load without sustaining damage. Some pipe materials exhibit characteristics of both rigid and flexible pipes such as bar wrapped concrete cylinder pipe, ductile iron pipe (EN 1295:2005).

Pipe Groups: Based on their Structural Flexibility

Rigid Pipe	Flexible Pipe	Semi-rigid Pipe
Rigid pipes deflect little; their load carrying capacity is derived from ring bending strength (as determined from crushing tests) and can be increased by bedding factors for various standardised bedding and surrounds. Rigid pipes tend to attract more load than flexible pipes, particularly in wide trench situations.	The flexibility of pipes is defined by the capability of pipes to deflect without showing signs of structural damage. Flexible pipes deflect under load in inverse relation to pipe stiffness and overall soil modulus. Flexible pipes derive their support primarily from passive soil resistance, which develops as the pipe ovalizes under vertical load and deflects horizontally into the side fill. The contribution of pipe stiffness is small.	Semi-rigid pipes also, like flexible pipes, deflect under load in inverse relation to pipe stiffness and overall soil modulus. Some pipe materials exhibit characteristics of both rigid and flexible pipes, primarily controlled by their diameters, and are referred to as semi-rigid (As per subsection 7.3.2 of the IS: 15155-2020, The structural properties of the pipe, together with the bedding and backfill, shall be designed to limit the deflection of pipe to not more than $D^2/(1.02 \times 10^5)$ mm, D is the nominal inside diameter of the pipe measured in millimetres.)

The modulus of elasticity is a measure of a material's stiffness and its inherent ability to deform when stress is applied. Metal piping–such as steel–has a high modulus of elasticity, which makes it very rigid. Depending on the type and intensity of the impact, metal piping such as copper is more likely to dent or crease, which can impede fluid flow. Small cracks may also form, weakening wall strength, limiting pressure capabilities, and increasing corrosion concerns. Plastic pipe has a very low modulus of elasticity, which means it has the flexibility to absorb the shock from a less severe impact and, to some extent, can bounce back to its original shape without any structural damage. Shock transferred to the connecting system will also be minimal. The main parameters that affect Young's modulus of material are: (a) temperature (with an increase in temperature, Young's modulus decreases) and (b) presence of impurity in the material like secondary phase particles, non-metallic inclusions, alloying elements, etc.

11.1.3 Selection of Pipe Material

11.1.3.1 Pipe Materials for Transmission Mains and Distribution Network

The determination of the suitability in all respects of the pipes and specials for any work is a matter of decision by the engineer concerned on the basis of requirements for the scheme, on techno-economic criteria. It is important to consider adaptability of pipes suiting the field conditions where bends are common in urban areas and need for providing service connections without damaging pipe characteristics. The Government policy nowadays is to provide service connections to all households, i.e., 100% households/premises.

Several technical factors affect the final choice of pipe material such as internal pressures, internal coating, coefficient of roughness, hydraulic and operating conditions, maximum permissible diameter, internal and external corrosion problems, laying and jointing, type of soil, special conditions, etc.

Selection of pipe materials should be based on the following considerations:

(a) The initial carrying capacity of the pipe and its reduction with use, defined, for example, by the Hazen-Williams coefficient C. Values of C vary for different conduit materials and their relative deterioration in service. They vary in size and shape to some extent.

(b) The strength of the pipe is measured by its ability to resist internal pressures and external loads.

(c) The life and durability of pipe is determined by the resistance of cast iron and steel pipe to corrosion; of concrete and AC pipe to erosion and disintegration and plastic pipe to cracking and disintegration.

(d) The ease or difficulty of transportation, handling and laying and jointing under different conditions of topography, geology, and other prevailing local conditions. This is particularly of hilly areas.

(e) The safety, economy, and availability of manufactured sizes of pipes and specials.

(f) The availability of skilled personnel in construction and commissioning of pipelines.

(g) The ease or difficulty of repairs during operations and maintenance.

(h) Soil chemistry – The influence of soil's chemical properties namely moisture contents, pH, temperature, soil resistivity, soil type, soil particle size, permeability, differential aeration, and sulphate-reducing bacteria, are reported to be a cause of failures of buried pipe due to reaction with the surface of unprotected buried pipes.

(i) Soil characteristics – The nature of the ground in which the piping is to be laid is to be invariably considered. There is alluvial soil, red soil, black cotton soil, arid/desert soil, laterite soil, saline and alkaline soil, peaty/marshy soil, forest soil, sub-mountain soil, and snowfields found in India as per the soil and land survey of India and according to their characteristics, certain metallic pipes and non-metallic pipes are suitably selected. In hilly areas, metallic pipes are preferably suitable due to undulated topography and rocky land.

(j) Soil resistivity – As per Clause 13.1.1 of IS 3043 (2018), the resistance to earth of a given electrode depends upon the electrical resistivity of the soil in which it is installed. This factor is important in deciding which of many protective systems to adopt for buried pipelines. As also described in SP:57 (1993 reaffirmed 2016) – Handbook on pipes and fittings for water supply (chapter 2, section 2) – corrosion resistance of pipes will not become a criterion for selection of pipe material, if proper protective coating is given.

The type of soil largely determines its resistivity and examples are given below in Table 11.1

Table 11.1: Soil Resistivity (as per clause 13.1.1 of IS 3043:2018 - Table 3)

Sl. No.	Type of Soil	Probable value in Ω.m (1 Ω.m = 100 Ω.cm)	Climatic Condition	
			Normal and High Rainfall (for Example, Greater than 500 mm a year)	Low Rainfall and Desert Condition (for Example, less than 250 mm a Year)
			Range of values encountered Ω.m (1 Ω.m = 100 Ω.cm)	Range of values encountered Ω.m (1 Ω.m = 100 Ω.cm)
(1)	*(2)*	*(3)*	*(4)*	*(5)*
i)	Alluvium and lighter clays	(500)	*	*
ii)	Clays (excluding alluvium)	10 (1000)	5 to 20 (500 to 2000)	10 to 100 (1000 to 10000)
iii)	Marls (for example, keuper marl)	20 (2000)	10 to 30 (1000 to 3000)	50 to 300 (500 to 30000)
iv)	Porous limestone (for example, chalk)	50 (5000)	30 to 100 (3000 to 10000)	-
v)	Porous sandstone (for example, keuper sandstone and clay shales)	100 (10000)	30 to 300 (3000 to 30000)	-
vi)	Quartzites, compact and crystalline limestone (for example, carboniferous marble, etc.)	300 (30000)	100 to 1000 (10000 to 100000)	-
vii)	Clay slates and slatey shales	1000 (100000)	300 to 3000 (30000 to 300000)	1000 (100000) upwards
viii)	Granite	1000 (100000)	-	-
ix)	Fossil slates, schists gneiss igneous rocks	2000 (200000)	1000 (100000) upwards	-
* Depends on water level of locality.				

The range of soil resistivity and Class of Soil (in terms of corrosivity) as per IS 3043 (2018) is given in Table 11.2 below:

Table 11.2: Soil Resistivity and Class of soil (in terms of Corrosivity) (as per Clause 13.6.1 of IS 3043:2018 – Table 4)

Sl. No.	Range of Soil Resistivity in Ω.m (1 Ω.m = 100 Ω.cm)	Class of Soil (in terms of corrosivity)
(1)	(2)	(3)
i)	Less than 25 Ω.m (2500 Ω.cm)	Severely Corrosive
ii)	25-50 Ω.m (2500-5000 Ω.cm)	Moderately Corrosive
iii)	50-100 Ω.m (5000-10000 Ω.cm)	Mildly Corrosive

Sl. No.	Range of Soil Resistivity in Ω.m (1 Ω.m = 100 Ω.cm)	Class of Soil (in terms of corrosivity)
iv)	Above 100 Ω.m (10000 Ω.cm)	Very Mildly corrosive)

(k) Pipes Mechanical properties: Certain mechanical properties are also considered while selecting pipe material for a specific service. They are:

- **Modulus of Elasticity (Young's Modulus)** – the ratio of stress to strain and measured using tension tests.
- **Elastic range –** Material returns to its original shape after load is released.
- **Plastic range –** Material is permanently deformed even after the load is released.
- **Yield Strength** – It defines the transition from elastic to plastic phase and it establishes the limiting value at which this transition occurs.
- **Ultimate Tensile Strength** – It defines the limit to which any further addition of load under constant strain would arrest the specimen elongation or thinning and would result in its failure.
- **Ductility** – expressed in the elongation of a specimen and its reduction in the cross-sectional area before its failure. Established by measuring specimen length before elongation and minimum diameter before failure.
- **Hardness** – Ability of a material to resist deformation. Hardness is tested by Brinell or Rockwell Hardness tests, both of which are indentation-type tests.
- **Toughness** – Ability of a material to resist sudden and brittle fracture due to the rapid application of loads. Measured using the Charpy V-Notch test.

In addition to the above points, while selecting the pipe and pipe materials, health related issues should also be kept on high priority. Many of the materials used for manufacturing pipe are found to cause certain health problems which needs to be properly examined before selection of pipe and pipe materials. The life and durability of the pipe depends on several factors including inherent strength of the pipe material, the manufacturing process along with quality control, handling, transportation, laying and jointing of the pipeline, surrounding soil conditions and quality of water.

Normally, the design period of pipelines is considered as 30 years for urban areas. Considering the design period, a suitable pipe material shall be selected which also best fits into all other selection criteria. Where the pipelines have been manufactured properly as per specifications, designed, and installed with adequate quality control and strict supervision, pipes have lasted more than the designed life provided the quality of water is non-corrosive. However, pipeline failures for various pipe materials even before the expiry of the designed life, have been reported probably due to lack of rigid quality control during manufacture and installation, improper design, presence of corrosive waters, corrosive soil environment, improper bedding, and other relevant factors. As pipelines are reticulated systems, the combined quality of pipes and pipelines arising out of quality pipe manufacture and sound installation, laying and jointing with strict supervision, standard jointing, bedding, backfilling and hydraulic pressure testing as per codes will determine the service delivery and life of a pipeline.

The metallic pipes are being provided with internal lining, either with cement mortar or epoxy or food grade compatible material so as to reduce corrosion, increase smoothness, and prolong the life. Lined metallic pipelines are expected to last beyond the normal design life of 30 years. However, the relative age of such pipes depends on the thickness and quality of lining available for corrosion. The cost of the pipe material and its durability or design life are the two major governing factors in the selection of the pipe material. It is necessary to carry out a detailed economic analysis before selecting a pipe material.

Underground metallic pipelines may require protection against external corrosion depending on the soil environment and corrosive ground water. Protection against external corrosion for MS pipes is provided with guniting with proper cement composition or epoxy coating, inside cement mortar lining with proper thickness, or hot-applied coal tar asphaltic enamel reinforced with fibre glass fabric yarn, non-corrosive sleeves, etc. Resistivity survey of the pipe alignment is important for external corrosion protection.

11.1.3.2 Health Aspects

While selecting the pipe material for drinking water supply systems, the following need to be kept in consideration:

- A leaking distribution system increases the likelihood of safe water leaving the source or treatment facility becoming contaminated before reaching the consumer. The pipe shall be strong enough to withstand external and internal forces without any damage.
- The pipe material or inner lining shall not have any constituent which may be unsuitable for human consumption especially the inner surface.
- Certain pipe materials which may be unsafe for human health are increasingly not utilised or being phased out such as lead, copper, etc.

11.1.3.3 Applicability

The applicability of different pipe materials varies with each site and the system requirements. The pipe material must be compatible with the soil and groundwater chemistry. The pipe material with proper coating must also be compatible with the soil structure and topography of the site, which affects the pipe location and depth, the supports necessary for the pipe fill material, and the required strength of the pipe material.

The following list shows background information to be used in determining what type of pipe best fits a particular situation:

- Maximum pressure conditions (force mains);
- Overburden, dynamic, and static loading;
- Lengths of pipe available;
- Soil conditions, soil chemistry, water table, stability;
- Joining materials required;
- Installation equipment required;
- Joint tightness/thrust control;
- Size range requirements;
- Field and shop fabrication considerations;
- Compatibility with existing systems;
- Thrust blocks, anchor blocks, valve chambers and other required structures to be included;
- Valves (number, size, and cost);
- Corrosion/cathodic protection requirements; and
- Maintenance requirements.

11.1.3.4 Installation Cost Consideration

Installation costs make up a major part of the total cost of a project. Differences in the cost of the actual pipe do not change the total cost of the project much. However, the following factors should be considered concerning installation costs and the choice of pipe:

- Weight of the pipe: A pipe that is lightweight can be handled easier and faster.
- Ease of assembling: Push-on joints can be assembled much faster than bolted joints.
- Pipe strength: If one type of pipe requires special bedding to withstand external pressures while another pipe does not, the choice can impact installation costs significantly.

11.1.3.5 Check List of Selection of Pipe Material

The life and durability of the pipe depends on several factors including inherent strength of the pipe material, the manufacturing process along with quality control, handling, transportation, laying and jointing of the pipeline, surrounding soil conditions and quality of water. Normally, the design period of pipelines is considered as 30 years. Where the pipelines have been manufactured properly as per specifications, designed and installed with adequate quality control and strict supervision, some of them have lasted more than the designed life provided the quality of water is non-corrosive. However, pipeline failures for various pipe materials even before the expiry of the designed life have been reported probably due to lack of rigid quality control during manufacture and installation, improper design, presence of corrosive waters, corrosive soil environment, improper bedding and other relevant factors. Lined metallic pipelines are expected to last beyond the normal design life of 30 years. However, the relative age of such pipes depends on the thickness and quality of lining available for corrosion. The cost of the pipe material and its durability or design life are the two major governing factors in the selection of the pipe material. The pipeline may have very long life but may also be relatively expensive in terms of capital and recurring costs and, therefore, it is very necessary to carry out a detailed techno economic analysis before selecting a pipe material. As pipelines are reticulated systems, the combined quality of pipes & pipelines arising out of quality pipe manufacture and sound installation, laying and jointing with strict supervision, standard jointing, bedding, back-filling and hydraulic pressure testing as per codes will determine the service delivery and life of a pipeline. And the manufactured quality of pipes for quality, strength and durability are not only the criteria for its service performance. The metallic pipes are being provided with internal lining either with cement mortar or epoxy or food grade compatible material so as to reduce corrosion, increase smoothness and prolong the life. The metallic pipes are being provided with internal lining either with cement mortar or epoxy so as to reduce corrosion, increase smoothness and prolong the life. Underground metallic pipelines may require protection against external corrosion depending on the soil environment and corrosive ground water. Protection against external corrosion is provided with cement mortar guniting or hot applied coal-tar asphaltic enamel reinforced with fiberglass fabric yarn, non-corrosive sleeves, etc. The structural design and indeed the choice of pipe material will be governed by factors such as the hydraulics of the flow system and the prevailing local conditions. Usually, the capacities of the commercial pipes vary considerably from theoretical values. It is, therefore, a common practice to design pipeline system for maximum discharges at non-silting and non-erodible velocities to minimize friction losses. The local conditions will vary from place to place and the magnitude will depend on factors such as soil type, dissolved chemicals in water, traffic, etc. However, generally, the pipe material and size of the pipe for a given scheme is determined by hydraulics and economic factors. As such, the determination of the suitability in all respects of the pipeline for any work is a matter of decision by the engineer concerned on the basis of the requirements for the scheme. However, selection of pipe materials must be based on the following main considerations including other criterion explained in this sub chapter.

(a) The initial carrying capacity of the pipe and its reduction with use, defined, for example, by the Hazen-Williams coefficient C. Values of C vary for different conduit materials and their relative deterioration in service. They vary with size and shape to some extent

(b) The strength of the pipe as measured by its ability to resist internal pressures and external loads.

(c) The life and durability of pipe as determined by the resistance of different pipes.

(d) The ease or difficulty of transportation, handling and laying and jointing under different conditions of topography, geology and other prevailing local conditions.

(e) The safety, economy and availability of manufactured sizes of pipes and specials.

(f) The availability of skilled personnel in construction and commissioning of pipelines.

(g) The ease or difficulty of operations and maintenance.

(h) Health factor: In addition to the above points, while selecting the pipe and pipe materials, health related issues should also be kept on high priority.

The selection of pipe material shall be done based on nature of pipe materials and field conditions.

11.1.3.6 Quality Monitoring and Implementation of Pipeline Projects

Water utilities often procure pipes from one manufacturer/ supplier under one contract, procure the valves and fittings from another manufacturer/ supplier under another contract and have them installed under another contract rather than entrusting the entire work of manufacture, Supply, Laying, Jointing, Testing and Commissioning of pipelines to a single agency. This procedure is resorted to on the plea that it results in economy and saves time.

It is seen that wherever single contracts are not awarded for the entire work of Manufacture, Supply, Laying, Jointing, Testing and Commissioning of pipelines to a single agency, the responsibility for performance of the pipelines could not be assigned to any particular agency. Time delays if any, in procurement of fittings and valves will also affect the completion of the contract and also results in cost overruns. Quite often, at the time of commissioning, deficiencies are noticed which might be due to failure at the manufacturing stage or due to transportation handling, or laying/jointing defects or failure of fittings and valves.

Hence it is desirable that all pipeline contracts are awarded on a single contract responsibility so that quality assurance at various stages of manufacture, supply, delivery, laying, jointing and testing is taken care of by a single agency and the timely completion also rests with a single agency; this may result in receipt of competitive offers and hence results in economy. Further, the water utility's time and resources which otherwise are spent in monitoring the performance of several small contracts can be better utilised for quality management of the contract. This may ensure economy by timely completion and quality construction.

However it is necessary that the specifications for single contract responsibility have to be comprehensive and provide for penalty in delays so that the time and cost over runs can be avoided. There will be several site specific conditions and circumstances for the pipeline installations which vary to such an extent that it is very difficult to recommend a simple/ single all inclusive set of specifications for the pipeline contracts.

To make the contractor accountable, the aforementioned recommendations for quality monitoring and implementation of Pipeline Projects shall be adhered to so as to avoid any delay in implementation and ensure quality of pipe material.

However, while contract for laying of pipeline works are awarded this should include a clause on the defect liability provisions for making the defect good during the defect liability period or the entire contract period whichever is more as per the contract model (EPC/BOT, etc.) including retaining a security bond equivalent to a certain percentage of the tender value (usually 1% to 10% relevant to the contract value) for the agreed performance guarantee during the entire liability period.

The following check list for drafting specifications for Manufacture, Supply, Laying, Jointing, Testing and commissioning of pipelines for procurement through a single agency is as follows. Judicious selection of items which cover cross country or city installations is required.

Check List for Specifications for Manufacture, Supply, Laying, Jointing, Testing and Commissioning Pipelines

PART I Procurement

Section 1 - General

1.1 Scope of work

1.2 Definitions of client, contractor, engineer etc.

1.3 Drawings and documents referred to

1.4 Reference Standards

1.5 Penal clauses for failure to meet the time schedule & performance standards and requirements.

1.6 Basis for Prices; to include all pipes, fittings, valves, jointing materials, including labour, cost of factory testing, lining, coating, marking and all other incidental expenses for manufacture, transportation, insurance and delivery at site. (any exclusions/ inclusions may be clearly specified)

Section 2 - Detailed Requirements - Pipes

2.1 Material for pipes (standards for materials), manufacturing operations, testing and inspection

2.2 Diameter of pipe

2.3 Wall thickness/ other dimensions of the pipe

2.4 Class of pipe

2.5 Laying length

2.6 Pipe ends-flanged-socket/ spigot/plain

2.7 Special pipe lengths and special fittings

2.8 Working Pressures

2.9 Pipe lining and coating both for buried and exposed pipes

Section 3 - Transportation and delivery at site

3.1 Type of trucks used for transportation-length / weight

3.2 Handling equipment for loading and unloading

Section 4 - Field Joints for Pipes

4.1 Requirements for machined couplings/ ends

4.2 Flanged/joints, pitch circle, blots type, gasket quality

4.3 Welded joints-runs-thickness

PART II INSTALLATION

Section 1 - Instruction to Bidders

1.1 Procedure for invitation of bids

1.2 Instructions to bidders

1.3 Bidders proposal to include plan/ programme for construction

1.4 Agreement and performance bonds

Section 2 - General Specifications

2.1 Definitions

2.2 Scope of Work

2.3 Payment conditions

2.4 Statutory Requirements- Payment of wages-Policy-Environment control-safety

2.5 Personnel

Section 3 - Detailed Specifications

3.1 Time Schedule

3.2 Construction facilities - Right of way - storage space - interference with other services

3.3 Work and materials

3.4 Concrete

3.5 Excavation - Bracing of excavation - Safety to public - Disposal of excess material from excavation

3.6 Maintenance, removal and reconstruction of other interfering facilities

3.7 Safeguarding of excavations and protection of property

3.8 Backfill

3.9 Resurfacing of roads within city and outside

Section 4 - Pipes

4.1 Approval of drawings for laying

4.2 Distribution along trench

4.3 Preparation of bedding

4.4 Lowering and laying

4.5 Jointing

11.2 Cast Iron Pipes

11.2.1 General

Cast iron (CI) pressure pipes may be classified in two categories, i.e., vertically cast iron (IS:1537-1976, reaffirmed 2020) and centrifugally cast (spun) iron (IS:1536-2001, reaffirmed 2021) pipes for water and sewage. Vertically cast iron has been largely superseded by centrifugally cast (spun) iron type up to a diameter ranging from 80 mm to 1050 mm for socket and spigot pipes (class LA, A, and B) and flanged pipes with screwed flanges (Class B). Though the vertically cast iron pipe is heavy in weight, low in tensile strength, and liable to defects of inner surface, it is widely used because of its good lasting qualities.

Cast Iron pipes and fittings are being manufactured in this country for several years. Due to its strength and corrosion resistance, CI pipes can be used in corrosive soils and for waters of slightly aggressive character. It is preferable to have coating inside and outside of the pipe.

Vertically cast iron pipes are manufactured by vertical casting in sand moulds. The metal used for the manufacture of this pipe is not less than grade 15. The pipes shall be stripped with all precautions necessary to avoid wrapping or shrinking defects. The pipes shall be such that they could be cut, drilled, or machined. Cast iron flanged pipes and fittings are usually cast in the larger diameters. Smaller sizes have loose flanges screwed on the ends of double spigot-spun pipe.

The method of cast iron pipe production used universally today is to form pipes by spinning or centrifugal action. Compared with vertical casting in sand moulds, the spun process results in faster production, longer pipes with vastly improved metal qualities, smoother inner surface and reduced thickness and consequent lightweight.

Centrifugally cast iron pipes are available in diameters from 80 mm to 1050 mm and are covered with protective coatings. Pipes are supplied in 3.66 m and 5.5 m lengths and a variety of joints are available including socket and spigot and flanged joints. The CI pipes have been classified as LA, A, and B according to their thickness. Class LA pipes have been taken as the basis for evolving the series of pipes. Class A allows a 10% increase in thickness over class LA. Class B allows a 20% increase in thickness over class LA. For vertically cast pipes, Class LA has not been taken as standard. For special uses, Classes C, D, E, etc. may be derived after allowing corresponding increases of thickness of 30, 40, 50 per cent, etc., over Class LA.

When the pipes are to be used for conveying potable water, the inside coating shall not contain any constituent soluble in water or any ingredient which could impart any taste or odour or impart health hazard.

Experiments in centrifugal casting of iron pipes were started in 1914 by a French Engineer which ultimately resulted in the commercial production of spun pipes. Spun pipes are about three-fourths of the weight of vertically cast pipes of the same class. The greater tensile strength of the spun iron is due to close grain allowing use of thinner wall than for that of a vertically cast iron pipe of equal length. It is possible by this process to increase the length of the pipe whilst a further advantage lies in the smoothness of the inner surface.

11.2.2 Laying and Jointing

11.2.2.1 Laying

Before laying the pipes, the detailed map of the area showing the alignment, sluice valves, scour valves, air valves, and fire hydrants along with the existing intercepting sewers, telephone and electric cables, and gas pipes will have to be studied. Care should be taken to avoid damage to the existing sewer, telephone and electric cables, and gas pipes. The pipeline may be laid on the side of the street where the population is dense. Pipes are laid underground with a minimum cover of 1 metre on the top of the pipe.

Laying of pipes for water supply system has been generally governed by respective Indian Standards (IS) as well as the regulations laid down by the Municipalities and Corporations in the States/UTs.

These regulations are intended to ensure proper laying of pipes giving due consideration to economy and safety of workers engaged in laying.

The pipes shall be straight. When rolled along two gantries separated by approximately two-thirds the length of the pipe to be checked, the maximum deviation from a straight line (in mm) shall not be greater than 1.25 times the length (in metres) of the pipes.

Excavation may be done by hand or by machine. The trench shall be so dug that the pipe may be laid to the required gradient and at the required depth. When the pipeline is under a roadway, a minimum cover of 1.0 m is recommended for adoption, but it may be modified to suit local conditions by taking necessary precautions. However, the structural strength of the pipe, based on dead load and live load over the pipe, should also be analysed. The trench shall be so braced and drained that the workmen may work therein safely and efficiently. The discharge of the trench dewatering pumps shall be conveyed either to drainage channels or to natural drains and shall not be allowed to be spread in the vicinity of the worksite.

The width of the trench at bottom between faces of sheeting shall be such as to provide not less than 200 mm clearance on either side of the pipe, except where rock excavation is involved. Additional width shall be provided at positions of sockets and flanges for jointing. Depths of pits at such places shall also be sufficient to permit finishing of joints.

Ledge rock, boulders, and large stones shall be removed to provide a clearance of at least 150 mm below and on each side of pipes in case of valves and fillings for pipes 600 mm in diameter or less, and 200 mm for pipes larger than 600 mm in diameter.

While unloading, pipes shall not be thrown down but may be carefully unloaded on inclined timber skids. Pipes shall not be dragged over other pipes and along concrete and similar pavements to avoid damage to pipes.

The pipes and fittings shall be inspected for defects and be rung with a light hammer, preferably while suspended, to detect cracks. Smearing the outside with chalk dust helps in the location of cracks. If doubt persists, further confirmation may be obtained by pouring a little kerosene on the inside of the pipe at the suspected spot. If a crack is present, the kerosene seeps through and shows on the outer surface. The pipe should be properly inspected on the site. Any pipe found unsuitable before and after laying, it shall be rejected.

11.2.2.2 Jointing

Several types of joints such as rubber gasket joint known as Tyton joint, mechanical joint known as screw gland joint, and conventional joint known as lead joint are used.
Joints are classified into the following three categories depending upon their capacity for movement.

a) Rigid joints

Rigid joints are those which admit no movement at all and comprise of flanged, welded and turned, and bored joints. Flanged joints require perfect alignment and close fittings are frequently used where a longitudinal thrust must be taken such as at the valves and meters. The gaskets used between flanges of pipes shall be of compressed fibre board or natural or synthetic rubber. Welded joints produce a continuous line of pipes with the advantage that interior and exterior coatings can be made properly and are not subsequently disrupted by the movement of joints.

b) Semi rigid joints

Semi rigid joint is represented by the spigot and socket with caulked lead joint. A semi rigid joint allows partial movement due to vibration, etc. The socketed end of the pipe should be kept against the flow of water and the spigot end of the other pipe is inserted into this socket. A twisted spun yarn is filled into this gap and it is adjusted by the yarning tool and is then caulked well. A rope is then placed at the outer end of the socket and is made tight fit by applying wet clay, leaving two holes for the escape of the entrapped air inside. The rope is taken out and molten lead is poured into the annular space by means of a funnel. The clay is then removed and the lead is caulked with a caulking tool.

Lead wool may be used in wet conditions. Lead covered yarn is of great use in repair work, since the leaded yarn caulked into place will keep back water under very low pressure while the joint is being made. Alternate yarn should also be explored and replaced with lead covered yarn in water supply due to adverse health impact on human being. Yarning or packing material shall also be considered for a) spun yarn, b) moulded or tubular natural or synthetic rubber rings, c) asbestos rope, or d) treated paper rope. These may be decided by the authority taking into consideration the quality of water.

c) Flexible joints

Flexible joints are used where rigidity is undesirable such as with filling of granular medium and when two sections cannot be welded. They comprise mainly mechanical and rubber ring joints or Tyton joints which permit some degree of deflection at each joint and are therefore able to stand vibration and movement. In rubber jointing, special type of rubber gaskets is used to connect cast iron pipe which are cast with a special type of spigot and socket in the groove, the spigot end being lubricated with grease and slipped into the socket by means of a jack used on the other end. The working conditions of absence of light, presence of water, and relatively cool uniform temperature are all conducive to the preservation of rubber and consequently, this type of joint is expected to last as long as the pipes. Hence, rubber jointing is to be preferred to lead jointing.

11.2.2.3 Fittings

All pipes, fittings, valves, and hydrants shall be carefully lowered into the trench by means of derrick, ropes, chain pulley blocks or other suitable tools and equipment depending on the weight and length of the pipes to prevent damage to pipe materials and protective coatings and linings.

All lumps, blisters and excess coating material shall be removed from socket and spigot end of each pipe and outside of the spigot and inside of the socket shall be wire-brushed and wiped clean and dry and free from oil and grease before the pipe is laid. After placing a length of pipe in the trench, the spigot end shall be centred in the socket and the pipe forced home and aligned to gradient. The pipe shall be secured in place with approved back fill material packed on both sides except at socket.

In general, the socket end should face the upstream while laying the pipeline on level ground however, the socket end should face the downstream subject to the strength of the joint and workmanship as per the guidelines. When the pipeline runs uphill, the socket ends should face the up gradient. When the pipes run beneath the heavy loads, suitable size of casing pipes or culverts may be provided to protect the casing of pipe. High-pressure mains need anchorage at dead ends and bends as appreciable thrust occurs which tend to cause draw and even “blow out” joints. Where thrust is appreciable, concrete blocks should be installed at all points where movement may occur.

Anchorages are necessary to resist the tendency of the pipes to pull apart at bends or other points of unbalanced pressure, or when they are laid on steep gradients and the resistance of their joints to longitudinal or shear stresses is either exceeded or inadequate. They are also used to restrain or direct the expansion and contraction of rigidly joined pipes under the influence of temperature changes. Anchor or thrust blocks shall be designed in accordance with IS: 5330-1984 reaffirmed 2020.

11.2.3 Testing of the Pipeline

After a new pipe has been laid, jointed, and backfilled, it shall be subjected to the pressure test and leakage test at a pressure to be specified by the authority for a duration of two hours.

After laying and jointing, the pipeline must be pressure tested to ensure that pipes and joints are sound enough to withstand the maximum pressure likely to be developed under working conditions.

11.2.3.1 Testing of Pressure Pipes

The field test pressure to be imposed should be not less than the maximum of the following:

a) 1.50 times the maximum sustained operating pressure.
b) 1.50 times the maximum pipeline static pressure.
c) maximum sustained operating pressure plus maximum surge pressure (in case of pumping mains);
d) sum of the maximum pipeline static pressure and the maximum surge pressure, subject to a maximum equal to the work test pressure for any pipe fitting incorporated;
e) testing pressure in accordance with the provisions of IS of relevant pipe material used.

- Operating pressure
  Maximum allowable operating pressure (MAOP) is the maximum pressure that can be safely operated by a pipeline. The thickness of the wall, pipe outer diameter, and specified minimum yield stress are used to calculate the MAOP of a pipe. Barlow's Formula relates the internal pressure that a pipe can withstand to its dimensions and the strength of its materials.

  The formula is $P = 2 \times T \times \frac{S}{D}$

  Where:

  P = pressure
  S = allowable stress
  T = wall thickness
  D = outside diameter

- Static pressure
  In hydrodynamics, it can be experimentally verified that each point of a fluid at rest exerts the same pressure around it in all directions. This pressure is called static pressure. This quantity is also known as hydrostatic pressure and does not depend on the velocity of flow, like dynamic pressure. The static pressure is part of the total pressure of a system, whose value still considers the quantities of the dynamic pressure and the gravitational potential energy of the system.

- Surge pressure
  A surge in pressure within a piping system, known as water or fluid hammer, occurs whenever the linear flow rate of fluid in pipe changes quickly – when pumps start or stop, valves open or close with quick acting actuation devices, or entrapped air moves within the system. The longer the pipeline and the faster the fluid is moving, the greater potential

for shock. Surges in pressure place stress on piping materials and joints and can cause physical movement of the piping system. Engineering designs must incorporate controls that can maintain surge pressures within the piping system's capabilities and eliminate or minimise physical motion of the system. It is very possible to have surge pressures twice as high as normal operating pressure. Long-term performance of the piping system can be affected by repetitive shock waves, potentially resulting in leaks and other costly damage. Some surge pressure problems result from poor piping system design – no matter what material is used for the system. Reducing the pipe size too quickly, for example, can result in surge pressure issues. The system may include 8-inch pipe when entering a tee and reduce down to two 3-inch pipes coming out of the tee. This type of situation creates a surge pressure inside the tee as the fluid has to greatly increase in linear velocity to push the same volume flow rate through a smaller cross-sectional flow area. Different materials perform differently in surge pressure situations depending upon their strength and elasticity. Understanding the material used in a piping system and designing the system to regulate pressure and fluid flow velocities according to its capabilities are important for the system's long-term performance. Proper sizing of pipe throughout the system, regulation of the speed with which valves and pumps actuate, and incorporation of surge dampening devices can limit the impact of hydraulic shock and keep the total system pressure within the design parameters. By combining quality piping and proper design, long-term performance can be ensured. Hydraulic Grade Line (HGL) – The hydraulic grade is the sum of the pressure head and elevation head. The hydraulic head represents the height to which a water column would rise in a piezometer. The plot of the hydraulic grade in a profile is often referred to as the hydraulic grade line, or HGL.

Invert level of pipe - Pipe invert elevation is the vertical distance between the inside-bottom of a pipe and the ground level.

If the visual inspection satisfies that there is no leakage, the test can be passed. Where the field test pressure is less than two-third the work test pressure, the period of test should be increased to at least 24 hours. The test pressure shall be gradually raised at the rate of 1 $kg/cm^2/min$ (0.1 $N/mm^2/min$). The field test pressure should wherever possible be not less than two-third work test pressure appropriate to the class of pipe except in the case of spun iron pipes and should be applied and maintained for at least four hours.

If the pressure measurements are not made at the lowest point of the section, an allowance should be made for the difference in static head between the lowest point and the point of measurement to ensure that the maximum pressure is not exceeded at the lowest point. If a drop in pressure occurs, the quantity of water added in order to re-establish the test pressure should be carefully measured. This should not exceed 0.1 litre per mm of pipe diameter per KM of pipeline per day for each 30 metre head of pressure applied. In case of gravity pipes, maximum working pressure shall be two-third work test pressure.

11.2.3.2 Procedure for Leakage Test

A leakage test shall be conducted concurrently with the pressure test. The allowable leakage, as per the clause 7.3.2 of BIS Code IS: 3114 (1994, reaffirmed 2019), during the pipe installation should not exceed the qL value (in $cm^3/hour$) calculated by the following formula:

$$qL = \frac{ND\sqrt{P}}{3.3} \tag{11.1}$$

Where

qL= Allowable leakage, cm^3/hour
N = No. of joints in the length of pipeline
D = Diameter, mm
P = The average test pressure during the leakage test, kgf/cm^2

Where any test of pipe laid indicates leakage greater than that specified as per the above formula, the defective pipe(s) or joint(s) shall be repaired/replaced until the leakage is within the specified allowance.

11.2.4 Advantages and Disadvantages

The advantages of pipe are:

- Good lasting qualities.
- Good strength, strong, and durable.
- Good corrosion resistance, if coated, and can be used in soils and for water of slightly aggressive character.
- Well suited for pressure mains and laterals where tapping is made for house connections.

The disadvantages of pipe are:

- heavy weight,
- short length,
- low tensile strength,
- liability to defect of inner surface,
- careful handling is required during transportation and jointing.

11.3 Ductile Iron Pipes

11.3.1 General

Ductile iron, also called nodular iron or spheroidal graphite iron, is characterised by the presence of graphite in nodular or spheroidal form in the resultant casting. It differs from cast iron by greater tensile strength and its significant elongation at break. Benefits of lower-cost production can be expected in a case where natural resources are used, allowing the disposal cost to be reduced. For a given nominal diameter, ductile iron pipes are duly designed with a larger internal diameter in order to reduce the head loss on energy pumping and the operation cost such as the electric power usage cost.

The DI pipes have excellent properties of machinability, impact resistance, high wear and tear resistance, high tensile strength, ductility, and corrosion resistance. DI pipes having same composition of CI pipe, it will have same expected life as that of CI pipes. The DI pipes are strong, both inner and outer surfaces are smooth, free from lumps, cracks blisters and scars. Ductile Iron pipes stand up to hydraulic pressure tests as required by service regulations. The DI pipes are available in the range of 80 mm to 1200 mm diameter; in lengths of 5.5 to 6 m. For diameter more than 1000 mm, necessary precautions should be adopted for proper jointing. For use and laying of DI pipes, IS 12288:1987, Reaffirmed 2022 may be referred.

11.3.2 Laying and Jointing

Laying

Pipes should be lowered into the trench with tackle suitable for the weight of pipes. For smaller sizes, up to 250 mm nominal bore, the pipe may be lowered by the use of ropes but for heavier pipes, either a well-designed set of shear legs or mobile crane should be used. When a lifting gear is used, the positioning of the sling to ensure a proper balance, should be checked when the pipe is just clear of the ground. If sheathed pipes are being laid, suitable wide slings or scissor dogs should be used. All construction debris should be cleared from the inside of the pipe either before or just after a joint is made.

It is recommended that above ground installations of spigot and socket pipes be provided with one support per pipe, the supports being positioned behind the socket of each pipe. Pipes should be fixed to the supports with mild steel straps so that axial movement due to expansion or contraction resulting from temperature fluctuation, is taken up at individual joints in the pipeline. In addition, joints should be assembled with the spigot end withdrawn 5 to 10 mm from the bottom of the socket to accommodate these thermal movements. Detailed specification on DI pipes may be referred at IS 8329: 2000 (Reaffirmed 2020).

The width of the trench at bottom between the faces of sheeting shall be such as to provide not less than 200 mm clearance on either side of the pipe except where rock excavation is involved. Trenches shall be of such extra width, when required, as will permit the convenient placing of timber supports, strutting and planking, and handling of specials.

Special consideration should be given to the depth of the trench. In agricultural land, the depth should be sufficient to provide a cover of not less than 900 mm so that the pipeline will not interfere with the cultivation of the land. In rocky ground, rough grazing or swamps, the cover may be reduced provided the water in the pipeline is not likely to freeze due to frost.

Where pipes are to be bedded directly on the bottom of the trench, it should be levelled and trimmed to permit even bedding of the pipeline. Where excavation is through rocks or boulders, the special bedding for DI pipes shall be as per BIS specifications.

For the purpose of backfilling, the depth of the trench shall be considered as divided into the following three zones from the bottom of the trench to its top:

a. Zone A: From the bottom of the trench to the level of the centre line of the pipe,
b. Zone B; From the level of the centre line of the pipe to level 300 above the top of the pipe, and
c. Zone C: From a level 300 mm above the top of the pipe to the top of the trench.

All backfill material shall be free from cinders, ashes, slag, refuse, rubbish, vegetable or organic material, lumpy or frozen material, boulders, rocks or stone or other material, which in the opinion of the authority, is unsuitable or deleterious. However, material containing stones up to 200 mm as their greatest dimension may be used in Zone C, unless specified otherwise herein.

Sand used for backfill shall be a natural sand, graded from fine to coarse. The total weight of loam and clay in it shall not exceed 10 per cent. All material shall pass through a sieve or aperture size 2.00 mm {IS: 2405 (Part II) -1980, Reaffirmed 2018}, and not more than 5 per cent shall remain on IS sieve or aperture size 0.63 mm.

Gravel used for backfill shall be natural gravel, having durable particles graded from fine to coarse in a reasonably uniform combination with no boulders or stones larger than 50 mm in size. It shall not contain excessive amount of loam and clay and not more than 15 per cent shall remain on a sieve of aperture size 75 micron.

For more details on backfilling and pipeline anchoring, BIS code IS 12288: 1987 (reaffirmed year: 2022)- Code of Practice for Use and Laying of Ductile Iron Pipes may be referred.

Jointing

All pipelines having unanchored flexible joints require anchorage at changes of direction and at dead ends to resist the static thrusts developed by internal pressure. Dynamic thrusts caused by flowing water act in the same direction as static thrusts.

Three main types of joints are used with ductile iron pipes and fittings (Refer BIS Code IS 8329-2000 (reaffirmed year: 2020) - Centrifugally Cast (Spun) Ductile Iron Pressure Pipes for Water, Gas and Sewage):

a) Socket and spigot flexible joints:
   (i) Push-on joint
   (ii) Mechanical joints
b) Rigid flanged joint
c) Restrained joint (bolted and boltless)

The spigot and socket flexible joint should be designed to permit angular deflection and axial movement to compensate ground movement and thermal expansion and contraction. They incorporate gasket of elastomeric material and the joints may be of the simple push-on type or mechanical joints. Flexible joints require to be externally anchored at all changes in direction such as at bends, etc., and at blank end to resist the thrust created by internal pressure and to prevent the withdrawal of spigots. Figure 11.2 shows flexible joint (push in type) and flanged joint.

In case of push-on joints, if any, they shall be suitably chamfered to facilitate smooth entry of spigot in the socket of the pipes or fittings fitted with rubber gasket. Push-on joint fittings are normally not used for sizes above DN 1 600. The material of rubber gaskets for use with mechanical joints and push-on joints shall conform to IS: 5382-2018–Rubber Seals—Joint Rings for Water Supply, Drainage and Sewerage Pipelines — Specification for Materials (Second Revision).

Figure 11.2: Flexible Joint (Push in type) and Flanged Joint

Rubber gaskets used with push-on joints or mechanical joints shall conform to (IS 5382-2018). Material of rubber gaskets for push-on mechanical or flanged joints shall be compatible with the fluid to be conveyed at the working pressure and temperature. Rubber gaskets for mechanical joint for conveyance of town gas may be suitably protected so that the elastomer does not come in direct contact with the gas. While conveying potable water, the gaskets should not deteriorate the quality of water and should not impart any bad taste or foul odour.

The dimensions and tolerances of the flanges of pipes and fittings shall be such, so as to ensure the interconnection between all flanged components (pipes, fittings, valves) of the same DN and PN and adequate joint performance. For screwed and welded on flanged pipes, the minimum classes for working pressure criteria are given in Clause 4.5 of IS 8329:2000, reaffirmed year 2020. Flanged joints are made on pipes having a machined flange at each end of the pipe. The seal is usually effected by means of a flat rubber gasket compressed between two flanges by means of bolts which also serve to connect the pipe rigidly. Gaskets of other materials, both metallic and non-metallic, are used for special applications.

In case of flange and mechanical joint casting, the flange shall be at right angle to the axis of the joint. The bolt holes shall be either cored or drilled. The centre of bolt holes circle shall be concentric with the bore circle and shall be located off the centre line. Where there are two or more flanges, the bolt holes shall be correctly aligned between them. The flanges shall be plain faced or with raised boss over the contact surface with a tool mark finishing having a pitch of 1 (±) 0.3 mm; serrations may be spiral or concentric.

An alternative and economical method of providing thrust restraint is the use of restrained joints. A restrained joint is a special type of push-on or mechanical joint that is designed to provide longitudinal restraint. Restrained joint systems function in a manner similar to thrust blocks, in so far as the reaction of the entire restrained unit of piping with the soil balances the thrust forces. Refer ISO 21052:2021 for determining the restraining length. In swamps or marshes where the soil is unstable or in other situations where the bearing strength of the soil is extremely poor, the entire pipeline shall be restrained joint system to provide adequate thrust restraint. As per the IS 8329-2000 (reaffirmed year 2020), for high-pressure mains where working pressure is substantial, depending on site condition, suitable flexible joint may be preferred where the joint is restrained against axial movement and the restraint joint may be considered in the transmission/gravity main where the working pressure is more than 8 kg/cm^2. Restrained joint pipe and fittings shall be used as carrier pipe for road crossings/railway crossings. In hilly areas where pipeline is laid on a slope, DI pipes with restrained joint shall also be used to avoid pipe slippage and joint separation. Figure 11.3 shows DI Boltless Restrained joint and Figure 11.4 shows DI bolted restrained joints.

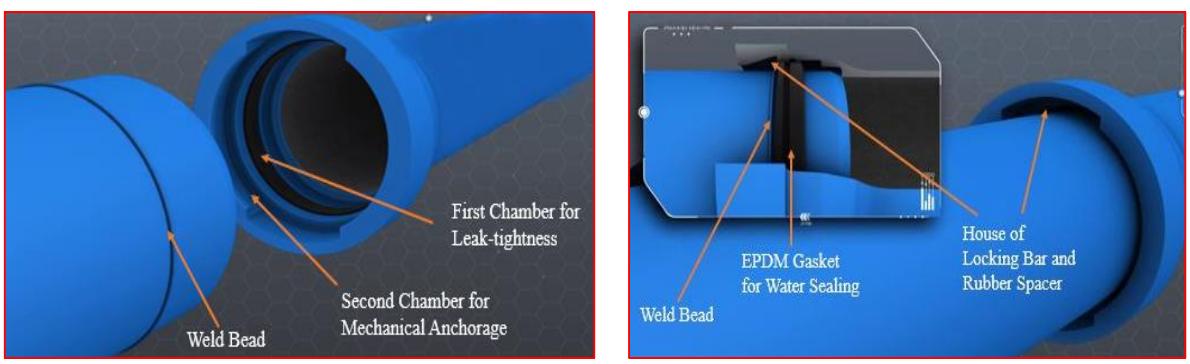


Figure 11.3: DI Boltless Restrained joint

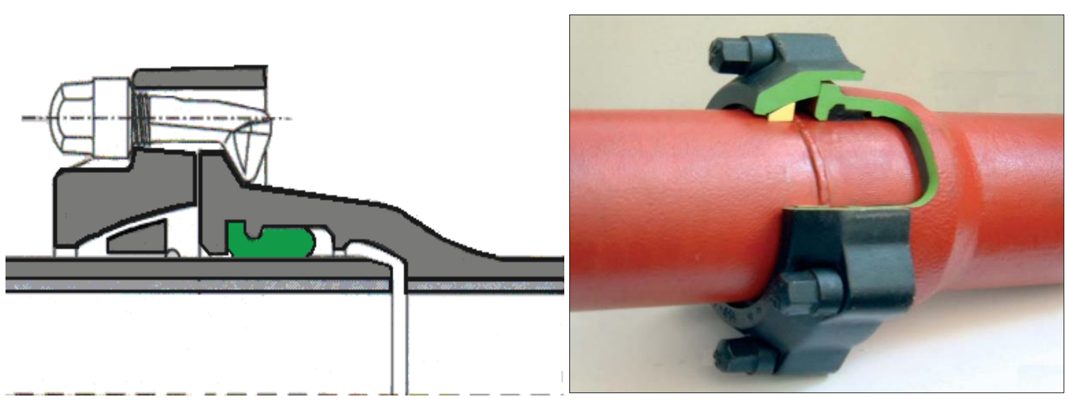


Figure 11.4: DI Bolted Restrained Joints

Procedure for jointing will vary according to the type of joint being used. Basic requirements for all types are:

a) Cleanliness of all parts,
b) Correct location of components,
c) Centralisation of spigot within socket, and
d) Strict compliance with manufacturer's jointing instructions.

Where the pipeline is likely to be subjected to movement due to subsidence or temperature variations, the use of flexible joints is recommended. A gap should be left between the end of the spigot and the back of the socket to accommodate such movement.

11.3.3 Fittings

The ductile iron fittings are manufactured conforming to (IS 9523-2000, reaffirmed 2020), for ductile iron fittings. The frequency of testing is related to the system of production and quality control at production level. The maximum batch size shall be 4 MT of crude castings, excluding the risers.

Mechanical tests shall be carried out during manufacturing by batch sampling system. The samples, being representative of finished product, are tested for tensile strength, elongation, and hardness to verify mechanical requirements. One test for castings produced within 24-hours period shall be adequate. The results obtained shall be taken as to represent all the fittings of all sizes made during that period.

For checking Brinell hardness test may be carried out on the test bar. Fittings shall be supplied in either 'as cast' condition or 'heat treated' condition.

For hydrostatic test, the fittings shall be kept under pressure for 10 seconds. They shall withstand the pressure test without showing any sign of leakage, sweating or other defect of any kind. The test shall be conducted before the application of surface coating.

When the pipe used under the conditions for which they are designed, in permanent or in temporary contact with water intended for human consumption, ductile iron pipes and their joints shall not have detrimental effects on the properties of the water for its intended use

11.3.4 Special Lining and Coatings for DI Pipes and Fittings

IS 8329 specifies bituminous or epoxy coating over Zinc Aluminium coating in ductile iron pipes. Industry has also come up with special coatings which are manufactured as per International Standards, like BS – EN, ISO, etc. The coatings are as follows.

11.3.4.1 Fusion-Bonded Epoxy (FBE) coating

Fusion-bonded epoxy (FBE) coating of steel materials is primer-less, one-part, heat-curable, thermosetting powdered epoxy coating which is designed to provide maximum corrosion protection to the substrate steel. It is a coating of very fast curing and thermosetting protective powder which utilises heat to melt and adhere the coating material to the steel substrate. It is based on specially selected epoxy resins and hardeners. The epoxy is formulated to meet the specifications related to protection of steel as an anti-corrosion coating. Heat cured FBE coatings are 100% solids consisting of thermosetting materials which achieve a high bond to metal surface because of a heat-generated chemical reaction. FBE is applied to coat DI pipes and fittings with powdered epoxy by fusion bonding process. The coatings enhance the corrosion-resistant properties of the piping system. It gives high gloss and smooth coatings with excellent adhesion. Difficult shapes can be coated evenly and provides enhanced corrosion restraint properties. FBE coating is widely used for coating of steel pipes, pipe fittings, pumps and valves used for the transmission of oil, gas, slurry, and water. This inert coating is suitable for extreme corrosive soils. Figure 11.5 shows application of fusion-bonded epoxy coating in various surfaces.

Figure 11.5: Application of fusion-bonded epoxy coating in various surfaces

11.3.4.2 Polyurethane (PU) Coating

A polyurethane coating is a polyurethane layer applied to the surface of a substrate for the purpose of protecting it. These coatings help protect substrates from various types of defects such as corrosion, weathering, abrasion and other deteriorating processes. The “special” coatings are designed to give greater external corrosion protection for pipes to be installed in aggressive soil conditions or for greater internal corrosion protection for pipes transporting highly corrosive liquids. The coating thickness is 700 microns as per EN 15189. The internal PU lining is applied in accordance with BS EN 15655:2018. Linings and coatings are applied in metallic pipelines are used for transporting abrasive and corrosive fluids as well as the pipes are laid in extremely corrosive environment. It is well suited for saline soil, coastal areas, ground with chemical contamination, refuse sites, farmyard, etc. Figure 11.6 shows polyurethane coated pipe.

Figure 11.6: Polyurethane coated pipe

11.3.4.3 High Alumina Cement Mortar Lining

Centrifugally applied inside metallic pipes in place of common cement mortar lining, high alumina cement mortar lining creates a mildly alkaline surface and protects the base metal of metallic pipe from corrosion and tuberculation and provides a tough surface to corrosive and abrasive fluids. High alumina cement should be conforming to BS EN 14647:2005.

11.3.4.4 Ceramic Epoxy Lining

Specialised lining applied inside the metallic pipeline which offers a hard and stable surface with high abrasion resistance. These linings are used for sewage conveyance or ash slurry conveyance or for conveying corrosive fluids.
The above is in general used for carrying sewage.

11.3.5 Testing of the Pipelines

After a new pipeline is laid and jointed, hydraulic testing shall be done for:

a) mechanical soundness and leak tightness of pipes and fittings;
b) leak tightness of joints; and
c) soundness of any construction work, in particular, that of the anchorages.

Hydrostatic testing may be performed for the completed pipeline either in one length or in sections; the length of section depending upon:

(i) availability of suitable water,
(ii) number of joints to be inspected, and
(iii) difference in elevation between one part of the pipeline and another.

Where the joints are left uncovered until after testing, sufficient material should be backfilled over the centre of each pipe to prevent movement under the test pressure. Progressively as experience is gained, lengths of about 1.5 km or more, should be tested in one section, subject to consideration of length of trench which can be left open in particular circumstances.

As per ISO 10802: 2020, the test pressure at the lowest point of the test section shall be not less than the limit specified in a) or b), whichever is greater.

a) For PW ≤ 10 bar: PST = 1.5 × PW
b) For PW > 10 bar: PST = PW + 5
c) The maximum PW: PST = PMDc

Where,

PW: highest pressure that occurs at a time and a point in the pipeline when operating continuously under stable conditions, without surge;
PST: pressure to which a pipeline or a pipeline section is subjected for testing purposes;

PMDc: maximum operating pressure of the system or of the pressure zone fixed by the designer considering future developments including surge in case of surge is calculated (pumping and water mains).

If the test is not satisfactory, the fault should be found and rectified. Where there is difficulty in locating a fault, the section under test should be subdivided and each part tested separately. Methods employed for finding leaks include:

(i) Visual inspection of each joint if, not covered by the backfill;
(ii) Use of a bar probe to detect signs of water in the vicinity of joints, if backfilled;
(iii) Aural inspection using a stethoscope or listening stick in contact with the pipeline;
(iv) Use of electronic listening device which detects and amplifies the sound or vibrations due to escaping of water, actual contact between the probe and the pipe is not essential;
(v) Injection of a dye into the test water particularly suitable in water-logged ground; and

(vi) Introduction of nitrous oxide in solution into the test water and using an infrared gas concentration indicator to detect the presence of any nitrous oxide that has escaped through the leak.

11.3.6 Advantages and Disadvantages

The advantages of pipe are:

- High resistance against breakage due to impact;
- High tensile strength, comparable to that of mild steel so that the pipes can be used for higher working pressure and can counter water hammer effectively;
- Traditional corrosion resistance, comparable to that of cast iron
- Lighter in mass as compared to cast iron pipes;
- Strong material properties and flexibility of joints contribute to prevent leakages;
- A comprehensive range of fittings manufactured and available;
- Easy incorporation of branches, service connections, etc.
- Well-developed mains and leakage detection systems.

The disadvantages of pipe are:

- Centrifugally cast pipe coatings required for protection against aggressive external operating environments.
- Risk of stray current corrosion.

11.4 Galvanised Iron (GI) Pipes

11.4.1 General

Galvanised iron (GI) pipes are manufactured using mild steel strips of low carbon steel coils. The strips are passed through a series of fin rolls to give them a circular shape. The slit ends of the strips are then welded together by continuously passing high-frequency electric current across the edges. The welded steel pipes are then passed through sizing sections where any dimensional deviations are corrected. The pipes are then cut into desired lengths by automatic cutting machines. The tubes are then pressure tested for any leaks. The galvanisation and varnishing of pipes are done as per specific requirements. Further details may be referred to in IS 4736: 1986 (Reaffirmed 2021), IS 1239 (Part 1): 2004, Reaffirmed 2019 and IS 1239 (Part 2): 2011, Reaffirmed 2021. Although negative manufacturing tolerances of 8% or more have been mentioned for different types of steel tubes and sockets in thickness of pipes in relevant IS code, some of the pipe manufactured lots may be defective. Therefore, practically for safety of pipe these tolerances can be reviewed by ULB considering the importance of field conditions typically prevalent in their area. Alternatively, the design working pressure can be considered with suitable factor of safety.

The GI Pipes are generally used for distribution of treated or raw water. These pipes are cheaper, light weight, and easy to transport. The health aspects should be given highest priority before selecting the pipe material for drinking water supply purposes as the pipe material is highly corrosive. It is not preferable to use GI pipes for house service connection (HSC).

The GI pipes are available in size range of 15 mm nominal bore to 150 mm outside diameter. 15 mm nominal bore to 150 mm nominal bore in the different classes of tubes like light, medium and heavy depending on wall thickness, are distinguished by colour bands such as light tubes (yellow), medium tubes (blue) and heavy tubes (red).

11.4.2 Laying and Jointing

All screwed tubes are supplied with pipe threads conforming to IS 554:1999, Reaffirmed 2019. Tubes, as per the IS 1239 (Part 1): 2004, Reaffirmed 2019, are supplied screwed with taper threads and fitted with one socket having parallel thread. The socket conforms to all requirements of IS 1239 (Part 2): 2011, Reaffirmed 2021. Pipes are generally joined using screwed joints. All threads for screw joints shall be clean, machine cut, and all pipes shall be reamed before erection. Each length of pipe as erected shall be upended and rapped to dislodge dirt and scale. Screwed joint shall be made up with good quality thread compound and applied to the male thread only. After having been set up, a joint must not be backed off unless the joint is completely broken, the threads cleaned, and new compound applied. Flange joints may be used if necessary. For pipe laid under ditches, pipe bedding shall be compacted for the entire length of the pipe, good alignment shall be preserved, and fittings may be used where necessary. Tubulars, sockets, and fittings shall be galvanised before screwing.

11.4.3 Testing of the Pipelines

Hydrostatic test is carried out at a pressure of 5 MPa (50.98 Kg-f/cm^2) (1 MPa =10.197 Kg-f/cm^2) and the same is maintained for at least three seconds and shall not show any leakage in the pipe.

Each tube shall be tested for leak tightness as an in-process test at manufacturer's works either by hydrostatic test or alternatively by eddy current test. Mass of zinc coating shall be determined in accordance with IS 4736: 1986 (Reaffirmed Year: 2021). The test for uniformity of zinc coating shall be done in accordance with IS 2633: 1986 (Reaffirmed Year: 2021).

The adhesion of zinc coating on fittings shall be determined by pivoted hammer test in accordance with IS 2629:1985 (Reaffirmed 2021). The zinc coating shall be reasonably smooth and free from such imperfections as flux, ash and dross inclusions, bare patches, black spots, pimples, lumpiness, runs, rust stains, bulky white deposits, and blisters.

11.4.4 Advantages and Disadvantages

Advantages of pipe are:

- Higher durability and longevity;
- Weld consistency and integrity;
- Amenable to rigorous fabrication;
- Superior finish and anti-rust coating;
- Greater corrosion resistance on the outer surface due to galvanisation;
- Superior bend ability and ease of cutting and threading.

Disadvantages of pipe are:

- Galvanised iron should never be used underground unless properly covered, which can be inconvenient for many jobs.
- It often hides significant defects beneath the zinc coating on the steel.
- Galvanised iron pipes may contain lead, which corrodes quickly and reduces the lifespan of the piping.
- Galvanised iron may leave rough patches inside pipes, resulting in serious failures and stoppages that can be expensive to repair.
- Prone to corrosion and leakage at the ferrule point, due to exposure of steel surface due to drilling to water.
- Greater incrustation inside the pipe increases head loss.

11.5 Steel Pipes

11.5.1 General

Steel pipes shall be manufactured with the steel produced by the open hearth or electric furnace or one of the basic oxygen processes. Other process may also be used by agreement between the purchaser and the manufacturer. The pipes shall be manufactured by the processes of seamless pipes, electric resistance welded pipes, and submerged arc welded pipes. The thickness of the steel pipe is controlled due to the need to make the pipe stiff enough to keep its circular shape during storage, transport, laying, and also to take the load of trench backfilling and vehicles. Injurious defects in pipe wall, provided their depth does not exceed one-third of the specified wall thickness, shall be repaired by welding.

Steel pipes of smaller diameter can be made from solid bar sections by hot or cold drawing processes and these tubes are referred to as seamless. But the larger sizes are made by welding together the edges of suitably curved plates, the sockets being formed later in a press (IS 3589: 2001, Reaffirmed 2022). The thickness of a steel pipe is, however, always considerably less than the thickness of the corresponding vertically cast or spun iron pipe. Owing to the higher tensile strength of the steel pipes (IS 3589:2001, Reaffirmed 2022), it is possible to make steel pipe of lower wall thickness and lower weight. The minimum thickness of steel pipe shall be as per Table 5 of IS 3589. Specials of all kinds can be fabricated without difficulty to suit the different site conditions.

The IS 5822 (1994, reaffirmed 2019) code of practice for laying of electrically welded steel pipes for water supply covers the methods of laying electrically welded mild steel pipes of outside diameters 168.3 mm to 2032 mm (as covered in IS 3589: 2001, Reaffirmed 2022), laid either above ground or underground for water supply.

11.5.2 Laying and Jointing

11.5.2.1 Laying

(A) Soil Resistivity Survey

Before laying of MS pipes, soil resistivity survey should be carried out.

The corrosion of underground MS pipes is related to the soil resistivity. If soil resistivity is lower, there are chances of corrosion. The resistivity rating is given in Table 11.2 under subsection 11.1.3.1.

Soil resistivity testing with accurate collection of data is the best indicator of the corrosivity of the soil for buried metallic structures and has a significant impact on the design of cathodic protection systems. The most common test methodology for field collection of soil data is the Wenner four-pin method. When properly collected, and using appropriate analytical techniques, the soil resistance field data can provide an accurate assessment of soil resistivity values for use in designing an appropriate cathodic protection system. The corrosion rate of the water pipeline is an essential parameter to predict pipeline damage. If the soil resistivity is 5000 ohm.cm and less (Table 11.2 under subsection 11.1.3.1), proper anti-corrosion measures should be taken. For example, external coating and wrapping of mild steel pipes is recommended to be done according to the IS 10221 (2008), Reaffirmed 2021 – Clause 8.2 and other relevant clauses. The trench shall be so dug that the pipe may be laid to the required alignment and at required depth. When the pipeline is under a roadway, a minimum cover of 1.0 m is recommended, but it may be modified to suit local conditions by taking necessary precautions. The trench shall be shored, wherever necessary, and kept dry so that the workman may work therein safely, and efficiently. The discharge of the trench dewatering pumps shall be conveyed either to drainage channels or to natural drains and shall not be allowed to be spread in the vicinity of the worksite. Open-cut trenches shall be sheeted and braced as required by any governing state laws and municipal regulations and as may be necessary to protect life, property, or the work. For all pipelines laid above ground, provision for expansion and contraction on account of temperature variation should be made either by providing expansion joints at predetermined intervals or by providing loops where leakage through expansion joints cannot be permitted.

Trenching includes all excavation which is carried out by hand or by machine. The width of the trench shall be kept to a minimum, consistent with the working space required. At the bottom between the faces, it shall be such as to provide not less than 200 mm clearance on either side of the pipe. The bottom of the trench shall be properly trimmed to permit even bedding of the pipeline. For pipes larger than 1,200 mm diameter in earth and murum the curvature of the bottom of the trench should match the curvature of the pipe as far as possible. Where rock or boulders are encountered, the trench shall be trimmed to a depth of at least 100 mm below the level at which the bottom of the barrel of the pipe is to be laid and filled to a like depth with lean cement concrete or with a non-compressible material like sand of adequate depth to give the curved seating. For details on laying of welded steel pipe, IS 5822-1994, Reaffirmed 2019 may be referred to.

Anchors to be designed as per IS 5330 (1984, Reaffirmed 2020), should be provided on the pipeline at the position of line valves or sectionalising valves, at the blank flange, at the tapers and at the mid-point between two consecutive expansion joints, in the case of above ground pipeline. Supports should be designed to support the pipeline without causing excessive local stresses. Due allowance shall be made for the weight of water, hydrostatic head, frictional resistance at the supports, etc. Proper bearing surface, such as flat base, roller and rocker, should be provided where controlled movements are required.

The internal design pressure shall not be less than the maximum pressure to which the pipeline is likely to be subjected including allowance for surge pressure, if any. The pipe selected shall be strong enough to withstand the effect of partial vacuum corresponding to one-third the atmospheric pressure which may occur within the pipe and due to any pressure exerted by water or soil around it.

Buried steel pipelines are liable to external corrosion and should be protected by the use of suitable coatings and shall be in accordance with IS 10221: 2008, Reaffirmed 2021. Pipelines laid above ground are liable to atmospheric corrosion and should be adequately protected.

(B) PROTECTIVE COATINGS

It must be borne in mind, however, that steel mains need protection from corrosion, internally and externally. Against internal corrosion, steel pipes are given epoxy lining or hot-applied coal tar/asphalt lining or rich cement mortar lining at works or in the field by the centrifugal process.

i. Three-Layer Polyethylene (3 LPE) Coating

External three-layer polyethylene coating applied over steel pipes have the following layers as given in the Figure 11.7 below.

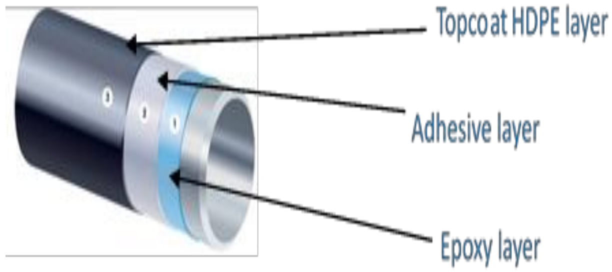


Figure 11.7: Special coating in Mild Steel pipe

a) **1st Primer layer of anti-corrosive fusion-bonded epoxy (FBE)**: In this layer, epoxy powder is spray-applied with electrostatic guns in typical thickness of 80 – 250 µm. The epoxy layer improves the bond with the steel base metal and arrests cathodic disbandment of the anti-corrosive coating with the metal surface.

b) **2nd Intermediate layer of copolymer adhesive**: Copolymer bonding layer extruded in typical thickness of 200 µm to bond the outer polyethylene layer to the epoxy prime coat.

c) **3rd Layer topcoat polyethylene**: Outer top layer comprising of polyethylene extruded in typical thickness of 1.8 – 3.5 mm, at service temperature up to 80 °C.

The three coats (3LPE) provide mechanical strength and anti-corrosive properties to the base metal of pipe.

Applicable Standards: 3LPE coating is applied as per International Standards (DIN 30670:2012, CAN.Z245.21.2018, ISO 21809-1:2018 specifications).
Application: 3LPE coating is applied in pipes when it is exposed to severe corrosive environment.

ii. Bituminous tape Coating (IS 10221-2008)

Anti-corrosive pipe wrapping tape is used for the protection of underground metallic pipelines from corrosion; fibre glass reinforced, bituminous tape comprising of thermofusible film on both sides, helps in protection against corrosion, UV and water ingress. The wrapping tape is applied by the help of a blow torch in outer surface of steel pipes. Figure 11.8 shows torch application for bituminous wrap coating and Figure 11.9 shows application of tape coating in pipeline.

Figure 11.8: Torch application for bituminous wrap coating

Figure 11.9: Application of tape coating in pipeline

iii. Internal Food Grade Solvent-Free Epoxy Coating

Food grade solvent-free liquid epoxy coating is applied inside water conveyance pipes. Apart from providing anti-corrosive protection, the coatings prevent chemical action, marine and algal growth inside the pipe surface, as well as reduce the friction losses in the pipe. Hot airless spray method is used for coating the internal surface of the pipe up to a thickness of 1.5 mm. Applicable standards are: Annex B for IS 3589:2001, Reaffirmed 2022, AWWA C210:2015. Figure 11.10 shows hot airless spray of food grade epoxy lining inside pipeline and Figure 11.11 shows internal food grade solvent-free epoxy coating.

Figure 11.10: Hot airless spray of Food grade epoxy lining inside pipeline

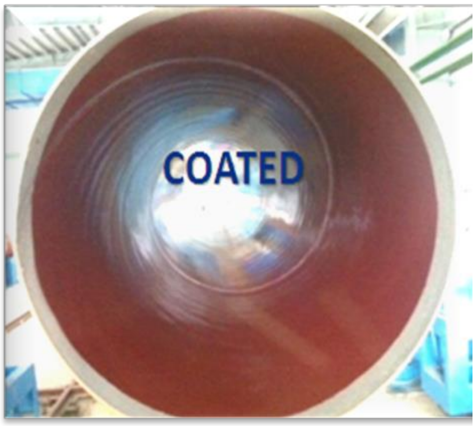


Figure 11.11: Internal Food Grade Solvent-Free Epoxy coating

iv. Guniting

The outer coating for underground pipeline may be in cement-sand guniting or hot-applied coal tar asphaltic enamel reinforced with fibre glass fabric yarn. The other materials may also be adopted for internal lining which should not provide adverse impact on health.

11.5.2.2 Jointing

The requirements for mild steel pipes up to 150 mm nominal diameter are covered in IS 1239 (Part 1): 2004, Reaffirmed 2019 and the requirements for mild steel and wrought steel fittings are covered in IS 1239 (Part 2): 2011, Reaffirmed 2021. These pipes can be joined by means of socket and screw or by welding. The requirements for steel pipes with diameters greater than 150 mm are covered in IS 3589: 2001, Reaffirmed 2022. The requirements for spiral welded pipes are given in IS 5504: 1997, Reaffirmed 2018. Higher diameter steel pipes and spiral welded pipes are joined by welding only.

Small size mild steel pipes have got threaded ends with one socket. They are lowered down in the trenches and laid to alignment and gradient. The jointing materials for this type of pipes are white lead and spun yarn. The white lead is applied on the threaded end with spun yarn and inserted into socket of another pipe. The pipe is then turned to tighten it. While laying, the pipes already stocked along the trenches are lowered down into the trenches with the help of chain pulley block. The formation of bed should be uniform. The pipes are laid true to the alignment and gradient before jointing. The ends of these pipes are butted against each other, welded and a coat of rich cement mortar is applied after welding.

Steel pipes may be joined with flexible joints or by welding but lead or other filler joints, hot or cold, are not recommended. The welded joint is to be preferred. In areas prone to subsidence, this joint is satisfactory but flexible joints must be provided to isolate valves and branches. When welding is adopted, plain-ended pipes may be joined by butt welds or sleeved pipes by means of fillet welds.

For laying long straight lengths of pipelines, butt joint technique may be employed. Steel pipes used for water supply include hydraulic lap welded, electric fusion welded, submerged arc welded and spiral welded pipes. The latter are being made from steel strip.

If the pipes are joined by a form of flexible joint, it provides an additional safeguard against failure. Steel pipes being flexible are best suited for high dynamic loading.

A. Expansion Joints

Para. 8.3.2 of the BIS code IS: 5822:1994 (Reaffirmed in 2019) – "Code of Practice for Laying of Electrically Welded Steel Pipes for Water Supply" mentions that for all pipelines laid above ground, provision for expansion and contraction on account of temperature variation should be made either by providing expansion joints at predetermined intervals or by providing loops where leakage through expansion joints cannot be permitted. Where expansion joints are provided, it is necessary to create restraining points on the pipeline to ensure proper functioning of these joints. The pipe laying work should preferably start from the restrained points on either side working towards centre where the expansion joint should be fitted last. Spacing of expansion joint depends on local conditions. Provision of expansion joint at intervals of 300 m on exposed steel pipeline is generally recommended. Expansion joints should always be provided between two fixed supports or anchorages.

B. Welded Joints Testing

Radiographic tests (non-destructive testing of fusion welded butt joints in steel pipes)

The BIS Code IS 4853 (1982, Reaffirmed 2020): covers the radiographic inspection of fusion welded butt joints in steel pipes up to the maximum size of 50 mm wall thickness, which may be referred to.

11.5.3 Fittings

For steel tubes, tubulars, and other wrought steel fittings, IS 1239 (Part I) (2004, Reaffirmed 2019) may be referred to.

11.5.4 Testing of the Pipelines

Before putting it into commission, the welded pipeline shall be tested both for its strength and leakage.

The manufacturer shall carry out the specified tests applicable to each type of pipe, and they shall, if required by the purchaser, supply a certificate stating that the pipes comply with the specified requirements.

Where the purchaser requires tests, the number of pipes on which mechanical tests shall be performed, shall be as follows:

(i) Up to and including 101.6 mm outside diameter – one pipe in each 400 tubes as made

(ii) Over 101.6 mm outside diameter – one pipe in each 200 pipes as made

If the number of samples specified in this clause, when applied to a particular order, necessitates a number of pipes which includes a fraction, the fraction shall be treated as unity.

Tensile test should be carried out in accordance with IS: 3601 (2006, Reaffirmed 2022).

The welded joints shall be tested in accordance with procedure laid down in the relevant BIS Standards. One test specimen taken from at least one field joint out of any 10 shall be subjected to test.

Each valved section of the pipe shall be slowly filled with clean water and all air shall be expelled from the pipeline through hydrants, air valves, and blow-offs fixed on the pipeline. Before starting the pressure test, the expansion joints should be tightened.

11.5.4.1 Pressure Test

The field test pressure to be imposed should be not less than the greatest of the following:

a) 1.5 times the maximum sustained operating pressure.
b) 1.5 times the maximum pipeline static pressure.
c) maximum sustained operating pressure plus maximum surge pressure (in case of pumping mains);
d) sum of the maximum pipeline static pressure and the maximum surge pressure, subject to a maximum equal to the work test pressure for any pipe fitting incorporated;
e) testing pressure in accordance with the provisions of IS of relevant pipe material used.

Where the field test pressure is less than two-thirds the factory test pressure, the period of test should be at least 24 hours. The test pressure shall be gradually raised at the rate of nearly 0.1 N/mm^2 per minute.

Each valve section of pipe shall be filled with water slowly and the specified test pressure, based on the elevation of lowest point of the linear section under test and corrected to the elevation of the test gauge, shall be applied by means of a pump connected to the pipe in a manner satisfactory to the authority.

Under the test pressure no leak or sweating shall be visible at all section of pipes, fittings, valves, hydrants, and welded joints. Any defective pipes, fittings, valves, or hydrants discovered in consequence of this pressure test shall be removed and replaced by sound material and the test shall be repeated until satisfactory to the authority.

11.5.5 Advantages and Disadvantages

The advantages of pipe are:

- Due to their elasticity, steel pipes adopt themselves to changes in relative ground level without failure and hence are very suitable for laying in ground liable to subsidence.
- Compared to cast iron pipes of the same size, steel pipes are lightweight. Steel pipes perform well in unusual circumstances, such as non-uniform bedding, settling soils, or when subjected to external loads due to their longitudinal strength.
- Easy installation of steel pipes is an advantage over cast iron pipes of similar strength. Steel pipes have smooth outer surfaces which makes installation using micro-tunnelling possible.
- Steel pipes have low frictional resistance to the flow of water. The thin walls of steel pipe generate a larger inside diameter which allows greater flow capacity.
- Risk of leaking mainly exists at the joints. Since steel pipes can be welded at the joints, the risk of leaking is much less.
- Service life of steel pipes depends on the rate of corrosion and abrasion. Impurities in water cause pipe abrasion to occur more rapidly.
- Reliability is a factor which can assure that pipe can tolerate any unprecedented event such as flood, soil movement, earthquake, etc. Steel is considered tough because of wide ductile range and ultimate stress resistance.

- A versatile pipe material will easily adapt to required modifications.If there are any changes in the bedding of the pipe, the beam strength of the steel can compensate.

The disadvantages of pipe are:

- Steel pipes required epoxy coating inside and 3LPE coating from outside and cathodic protection of whole pipeline.
- Welding joints for steel water pipe are complicated and require skilled labour.
- Steel pipe is susceptible to internal tuberculation and external corrosion, and is subjected to electrolysis, if not properly protected.
- Use of anti-corrosion products increases the price of production and maintenance of steel pipe.

Air vacuum valves are necessary in large diameter pipes to prevent collapse.

11.6 Asbestos Cement (AC) Pressure Pipes

11.6.1 General

Asbestos cement pressure (AC pressure) pipes are made of a mixture of chrysotile asbestos paste (only contains 8%-10%) and cement (50%), clay (30-35%) and fly ash, wood, pulp, etc., which are not considered harmful for human health. Even here the asbestos fibres are locked with cement matrix particles and there is no scope for its disintegration/spreading in the air in normal circumstances. The paste is compressed by steel rollers to form a laminated material of great strength and density. Its carrying capacity remains substantially constant as when first laid, irrespective of the quality of water. It can be drilled and tapped for house service connection through a saddle piece as per clause 9.1 and 9.1.1 of IS 6530 1972, Reaffirmed 2022. The hole of required size shall be drilled through the pipe and the boss (cushion) provided in the top strap. Ferrule piece shall be connected after making threads in the boss and pipe. Suitable rubber packing shall be used between the straps and the pipe to provide cushioning as well sealing against leakages.

AC pressure pipes are manufactured from classes 10 to 25 and nominal diameters of 80 mm to 1000 mm with the test pressure of 10 to 25 Kg/cm^2. AC pressure pipe can meet the general requirements of water supply undertakings for rising main as well as distribution main. It is classified as class 10, 15, 20, and 25, which have test pressures 10, 15, 20, and 25 Kg/cm^2 respectively. Working pressures shall not be greater than 50% of test pressure for pumping mains and 67% for gravity mains. The BIS Code for the highest class of ACP pipe is class 25 having test pressure of 25kg/cm^2 and working pressure of 12.5 kg/cm^2, which is equivalent to Class K7 of Ductile Iron Pipe having working pressure of 12kg/cm^2.

ACP pipe being non-metallic, once buried, the metal detector is ineffective in locating such pipes, therefore, metallic locating tapes or copper wires can be placed alongside the ACP pipe without causing significant cost, to facilitate locating buried ACP pipes.

These pipes are not suitable for use in sulphate soils. Due to expansion and contraction of black cotton soil, usage of these pipes may be avoided as far as possible in black cotton (B.C.) soils, except where the depth of B.C. soil is clearly less than 0.9 metre below ground level.

Storage of AC pipes shall be done on firm level and clean ground and wedges shall be provided at the bottom layer to keep the stack stable. The stack shall be in pyramid shape or the pipes laid lengthwise and crosswise in alternate layers. The pyramid stack is advisable in smaller diameter pipes for conserving space in storing them. The height of the stack shall not exceed 1.5 m.

11.6.2 Laying and Jointing

11.6.2.1 Laying

The pipes shall have a minimum soil cover of 750 mm when laid under foot paths andsidewalks, 900 mm when laid under roads with light traffic or under cultivated soils, and 1250 mm when laid under roads with heavy traffic. When the soil has a poor bearing capacity and is subject to heavy traffic, the pipes shall be laid on a concrete cradle. An extra trench depth of 100 mm shall be provided for each jointing pit.

The pipes shall be lowered into the trenches either by hand passing or by means of two ropes. One end of each rope shall be tied to a wooden or steel peg driven into the ground and the other end shall be held by men which when slowly released will lowerthe pipe into the trench. The width of the trench should be uniform throughout the length and greater than the outside diameter of the pipe by 300 mm on either side of the pipe. The depth of the trench is usually kept 1 metre above the top of the pipe. For heavy traffic, a cover of at least 1.25 metre is provided on the top of the pipe.

In unstable soils, such as soft soils and dry lumpy soils, it shall be checked whether the soils can support the pipelines and if required, suitable special foundation shall be provided. In places where rock is encountered, a cushion of fine earth or sand shall be provided for a depth of 150 mm by excavating extra depth of the trench, if necessary, and the pipes laid over the cushion. Where the gradient of the bed slopes is more than 30°, it may be necessary to anchor a few pipes against their sliding downwards.

The excavation of the trench shall be so carried out that the digging of the trenches does not get far ahead of the laying operations. By doing this, the risk of falling of the sides and flooding of trenches shall be avoided. The walls of the trench shall be cut generally to a slope of ¼:1 or ½:1, depending on the nature of the soil. If the trench bottom is extremely hard or rocky or loose stony soil, the trench should be excavated at least 150 mm below the trench grade.

Prior to being placed in the trench, pipes should be visually inspected for evidence of damage as any damage to the pipe may impair its strength or integrity consequently. Before use, the inside of the pipes will have to be cleaned. The lighter pipes weighing less than 80 Kg can be lowered in the trench by hand. If the sides of the trench slope too much, ropes must be used. The pipes of medium weight up to 200 Kg are lowered by means of ropes looped around both the ends. One end of the rope is fastened to a wooden or steel stack driven into the ground and the other end of the rope is held by men and is slowly released to lower the pipe into the trench. After their being lowered into the trench they are aligned for jointing. The bed of the trench should be uniform. Utmost care must be taken while loading, transportation, unloading, stacking, and carrying to the site to avoid damage to the pipes. Figure 11.12 shows Cutting of AC Pressure Pipes and Figure 11.13 shows Laying of AC Pressure Pipes in Rocks.

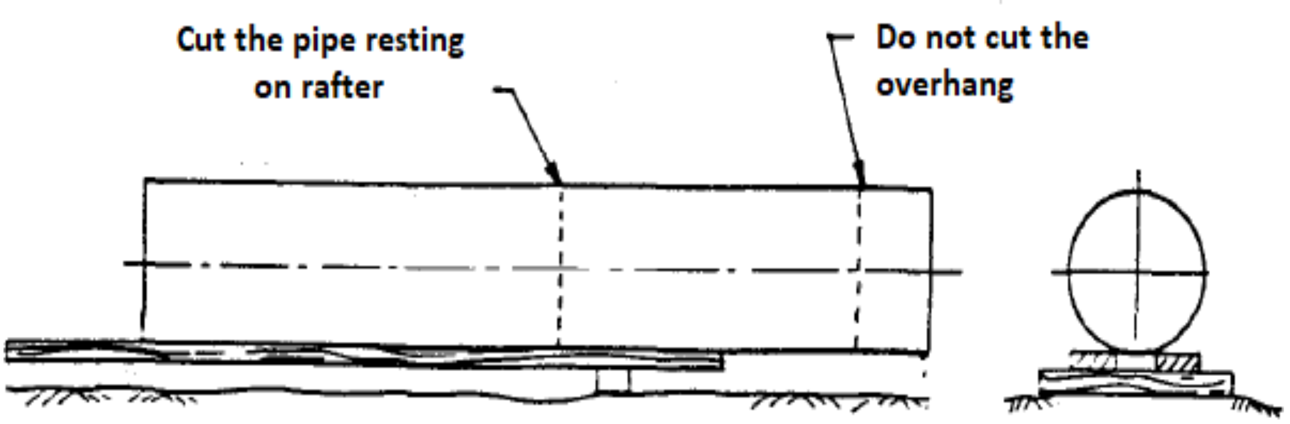


Figure 11.12: Cutting of AC Pressure Pipes

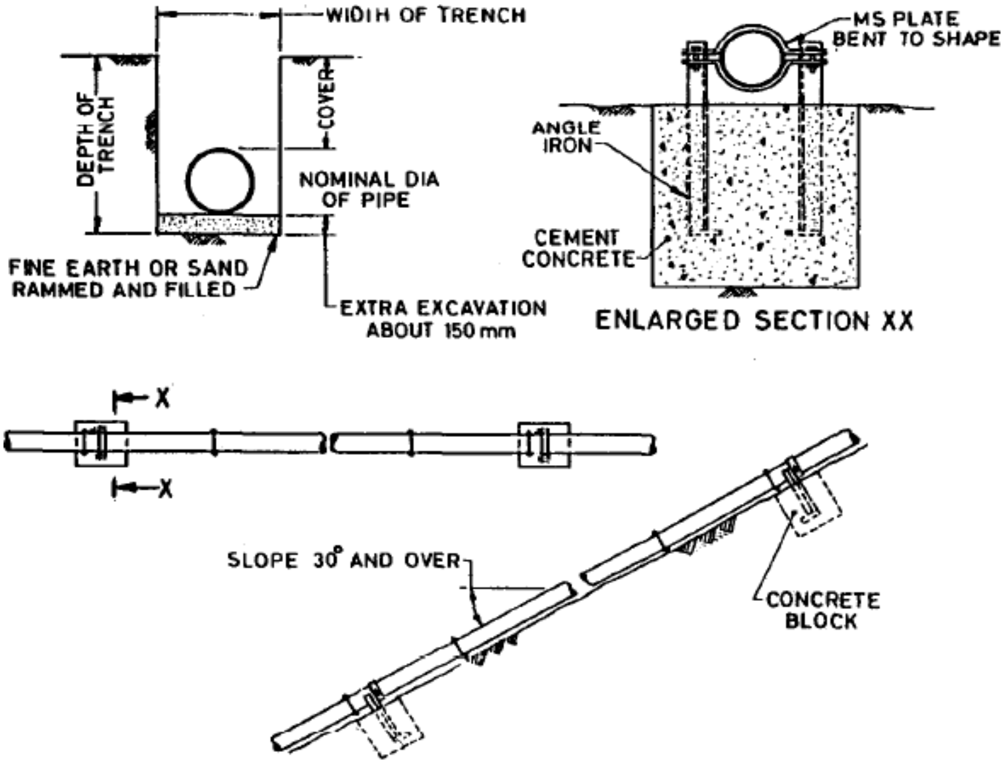


Figure 11.13: Laying of AC Pressure Pipes in Rocks

11.6.2.2 Jointing

Before commencing jointing, the pipes shall be cleaned; the joints and the ends of the pipe shall be cleaned, preferably with a hard wire brush to remove loose particles.

Two types of joints are normally provided with AC pressure pipes and they are:

a. Cast iron detachable (CID) joints (IS 8794 1988, Reaffirmed Year 2022)
b. AC coupling joints ((IS 1592: 2003 (Reaffirmed Year 2018))

a) Cast Iron Detachable Joints

This consists of two cast iron flanges, a cast iron central collar and two rubber rings along with a set of nuts and bolts for the particular joint. For this joint, the AC pipes should have flush ends. For jointing a flange, a rubber ring and a collar are slipped to the first pipe in that order; a flange and a rubber ring being introduced from the jointingof the next pipe. Both the pipes are now aligned, the collar centralised, and the joints of the flanges tightened with nuts and bolts. Figure 11.14 shows Cast Iron Detachable Joints.

b) Asbestos Cement Coupling Joint

This consists of an AC coupling and three special rubber rings. The pipes for these joints have chamfered ends. These rubber rings are positioned in the grooves inside the coupling, then grease is applied on the chamfered end and the pipe and coupling is pushed with the help of a jack against the pipe. The mouth of the pipe is then placed in the mouth of the coupling end and then pushed so as to bring the two chamfered ends dose to each other. Wherever necessary, change over from cast iron pipe to AC pipes or vice versa, should be done with the help of suitable adapters. IS 6530: 1972, Reaffirmed 2022, may be followed for laying AC pipes. Figure 11.15 shows Asbestos Cement Coupling Joint.

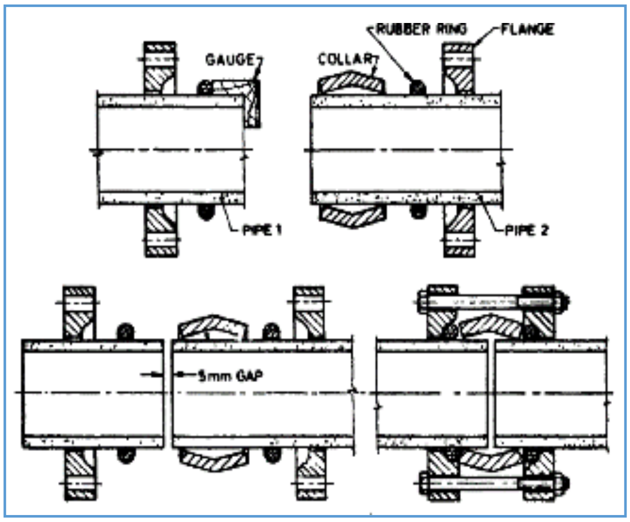


Figure 11.14: Cast Iron Detachable Joints

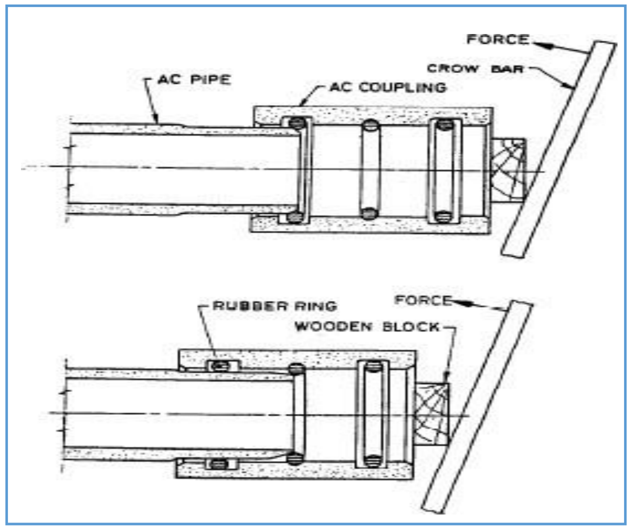


Figure 11.15: Asbestos Cement Coupling Joint

However, in the past, projects implemented with AC pipes joined with an inordinate use of Cast Iron Detachable (CID) joints in the main water distribution network (limited to be used only in T-points, bends, etc.) have not only resulted in huge capital cost due to higher cost of the joints than the pipe material, but also resulted in huge leakages at the joints over time, as Cast Iron joints are prone to corrosion. Therefore, confirming to BIS IS 1592:2003 (reaffirmed 2018), the main distribution network of the water supply scheme should be jointed with the AC coupling and three rubber special rings fitted inside the coupling as it provides an air- and water-tight grip with leak-proof joints. AC couplings are economical, non-corrosive, and easy to repair and maintain. During maintenance, the damaged pipe section can be easily replaced with a light hammer in the joints, the AC coupling can be broken, and the damaged pipe can be replaced with a new AC pipe. The couplings are also easily available as they are manufactured in India.

Although, the BIS Code IS 10299:1982 (Reaffirmed 2020) specifies the CI saddle pieces for service connection in Asbestos Cement Pressure Pipes, DI saddles with epoxy powder coating shall be used with AC pipe instead of the cast iron detachable (CID) saddle to avoid any corrosion or leakages.

11.6.3 Fittings

When a fitting is used to make a vertical bend, it shall be anchored to a concrete thrust block designed to have enough weight to resist the upward and outward thrust. Similarly, at joints deflected in vertical plane, it shall be ensured that the weight of the pipe, the water in the pipe, and the weight of the soil over the pipe provide resistance to upward movement. If it is not enough, ballast or concrete shall be placed around the pipe in sufficient weight to counteract the thrust.

Pipes on the slope need to be anchored only when there is a possibility of the backfillaround the pipe sloping down the hill and carrying the pipe with it. Generally, for slopesup to 30°, good well-drained soil carefully tampered in layers of 100 mm under and over the pipe, right up to the top of the trench, will not require anchoring.

Normally, when a pipeline is laid, a certain number of cast iron fittings such as tees, bends, reducers, etc., and special fittings such as air and sluice valves are required. All cast iron fittings shall be plain-ended to suit the outside diameter of asbestos cement pressure pipes and to the class and diameter of pipe manufactured. When using such cast iron fittings, they are joined by cast iron detachable joints only. For any cast iron specials having flanges, they are jointed in the pipeline with cast iron flange adaptors having one end flanged and the other plain-ended. However, DI fittings should be preferred over CI fittings in all the applications.

11.6.4 Testing of the Pipelines

After all sections have been jointed together on completion of section testing, a test on the complete pipeline should be carried out. Asbestos cement pipes always absorb a certain amount of water. Therefore, after the line is filled, it should be allowed to stand for 24 hrs before pressure testing, and the line shall be again filled. This test should be carried out at a pressure not less than the working pressure of the pipeline, care being taken to ensure that the pressure at the lowest point in the pipeline does not exceed the maximum. During the test, an inspection should be made of all work which has not been subjected to sectional tests. The test pressure shall be gradually raised at the rate of approximately 1 $kg/cm^3/min$. The duration of the test period, if not specified, shall be sufficient to make a careful check on the pipeline section. After the test has been completed, the trench shall be filled back.

The procedure for the pressure testing as adopted is as follows:

a) At a time, one section of the pipeline between two sluice valves is taken up for testing. The section usually taken is about 500 metres long.
b) One of the valves is closed and the water is admitted into the pipe through the other, manipulating air valves suitably. (If there are no sluice valves in between the section, the end of the section can be sealed temporarily with an end cap having an outlet which can serve as an air relief vent of for filling the fine as may be required. The pipeline after it is filled should be allowed to stand for 24 hours before pressure testing).
c) After filling, the sluice valve is closed and the pipe section is isolated.
d) Pressure gauges are fitted at suitable intervals on the crown into the holes meant for the purpose.
e) The pipe section is then connected to the delivery side of a pump through a small valve.
f) The pump is then operated till the pressure inside reaches the designed value which can be read from the pressure gauges fixed.
g) After the required pressure has been attained, the valve is closed and the pumpdisconnected.
h) The pipe is then kept under the desired pressure during inspection for any defect, i.e., leakages at the joints, etc. The water will then be emptied through scour valves and defects observed in the test will be rectified.

11.6.5 Advantages and Disadvantages

The advantages of pipe are:

- The inside surface of pipe is smooth.
- The joining of pipes is very good and flexible.
- The pipes are anti-corrosive and cheap in cost.
- Light in weight to handle and transport.
- Least storage cost as it can be stored in open space at worksite and also anti-theft due to its no resale value.

The disadvantages of pipe are:

- Not suitable for black cotton soil and sulphate contained soil.
- Not suitable for hilly hard rocky terrain.

11.7 Reinforced Cement Concrete Pipes (RCC)

11.7.1 General

RCC pipes used in water supplies are classified as P1, P2, and P3 with test pressures of 2.0, 4.0, and 6.0 Kg/cm^2 respectively. For use as gravity mains, the working pressure should not exceed two-third of the test pressure. For use as pumping mains, the working pressure should not exceed half of the test pressure.

Generally concrete pipes have corrosion-resistant properties similar to those of prestressed concrete pipes although they have their own features which significantly affect corrosion performance. Reinforced concrete pipes either spun (centrifugal spinning) or vibrated cast (vibratory processes) shall be designed such that the maximum tensile stress in the circumferential steel due to specified hydrostatic test pressure does not exceed the limit of 125 N/mm^2 (IS 458:2021) (1274.65 Kgf/cm^2) in the case of mild steel rods, 140 N/mm^2 in the case of hard-drawn steel wires and high strength deformed steel bars and wires. Centrifugally spun pipes are subjected to high rotational forces during manufacture with improved corrosion resistance properties. The line of development most likely to bring concrete pressure pipes into more general acceptance is the use of PSC pipes which are widely used to replace reinforced concrete pipes.

11.7.2 Laying and Jointing

11.7.2.1 Laying

The concrete pipes should be carefully loaded, transported, and unloaded avoiding impact. Free working space on either side of the pipe shall be provided in the trench which shall not be greater than one-third the diameter of the pipe but not less than 150 mm on either side. Pipes should be lowered into the trench with tackle suitable for the weight of pipes, such as well-designed shear slings with chain block or mobile crane. While lifting, the position of the sling should be checked when the pipe is just clear off the ground to ensure proper balance.

In general, the IS 783: 1985 (Reaffirmed Year: 2022) Code of Practice for Laying of Concrete Pipes may be referred to. Laying of pipes shall proceed upgrade of a slope. If the pipes have spigot and socket joints the socket ends shall face upstream. The pipes shall be joined in such a way to provide as little unevenness as possible along the inside of the pipe. Where the natural foundation is inadequate, the pipes shall be laid in a concrete cradle supported on proper foundation or any other suitably designed structure. If a concrete cradle is used, the depth of concrete below the bottom of the pipes shall be at least one-fourth the internal diameter of pipe with the range of 100 mm to 300 mm. It shall extend up to the sides of the pipe at least to a distance of one-fourth the diameter for larger than 300 mm.

The pipe shall be laid in the concrete bedding before the concrete has set. Trenches shall be back filled immediately after the pipe has been laid to a depth of 300 mm above the pipe subject to the condition that the jointing material has hardened (say 12 hours at the most). The backfill material shall be free from boulders, roots of trees, etc. The tamping shall be by hand or by other hand operated mechanical means. The water content of the soil shall be as near the optimum moisture content as possible. Filling of trench shall be carried on simultaneously on both sides of the pipe to avoid development of unequal pressures. The back fill shall be rammed in 150 mm layers up to 900 mm above the top of the pipe.

Where gradient steeper than 1 in 6 is contemplated, consideration should be given to the construction of suitable transverse anchor blocks. For gradients between 1 in 7 and 1 in 12, the need for transverse anchor blocks will depend on ground conditions. For slopes flatter than 1 in 12, there is seldom a need to provide anchor blocks.

11.7.2.2 Jointing

Joints may be of any of the following types:

(i) Bandage joint
(ii) Spigot and socket joint (rigid and semi-flexible)
(iii) Collar joint (rigid and semi-flexible)
(iv) Flush joint (internal and external)

In all pressure pipelines, the recesses at the ends of the pipe shall be filled with jute braiding dipped in hot bitumen. The quantity of jute and bitumen in the ring shall be just sufficient to fill the recess in the pipe when pressed hard by jacking or any other suitable method.

The number of pipes that shall be jacked together at a time depends upon the diameter of the pipe and the bearing capacity of soil. For small pipe up to 250 mm diameter, six pipes can be jacked together at a time. Before and during jacking, care should be taken to see that there is no offset at the joint.

Loose collar shall be set up over the joint so as to have an even caulking space all round and into this caulking space shall be rammed 1:1.5 mixture of cement and sand just sufficiently moistened to hold together in the form of a clod when compressed in the hand. The caulking shall be so firm that it shall be difficult to drive the point of a penknife into it. The caulking shall be employed at both the ends in a slope of 1:1. In the case of non-pressure pipes the recess at the end of the pipes shall be filled with cement mortar 1:2 instead of jute braiding soaked in bitumen. It shall be kept wet for 10 days for maturing.

A. Rigid Joints

In this, the water seal is affected by cement mortar or similar material which will not allow any movement between the two pipes.

Socket and Spigot Joint: The annular space between socket and spigot is filled with cement mortar (1:2). This joint is used for low-pressure pipeline. Figure 11.16 shows spigot and socket joint.

Collar Joint: Collars of 150 to 200 mm wide cover the joint between two pipes. A slightly damp mixture of cement and sand is rammed with caulking tool. Figure 11.17 shows collar joint (rigid).

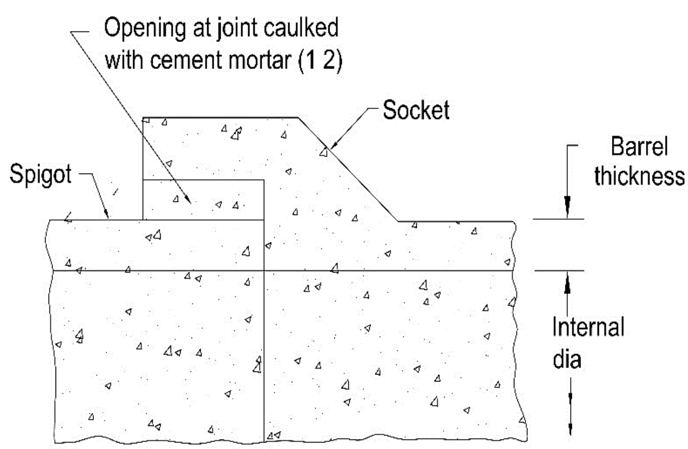


Figure 11.16: Spigot and Socket Joint

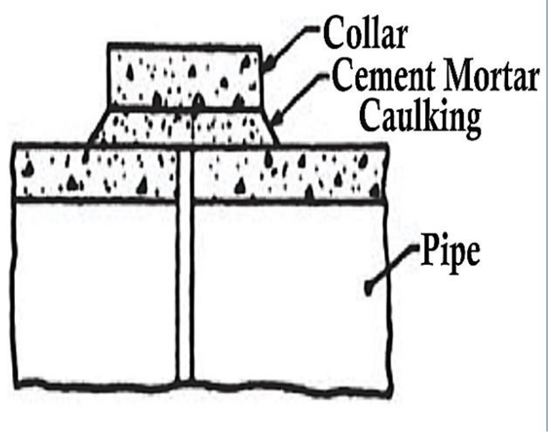


Figure 11.17: Collar Joint (Rigid)

Flush Joint

Internal Flush joint – This joint is generally used for culvert pipes of 900 mm diameter and over. The ends of the pipes are specially shaped to form a self-centring joint with an internal jointing space 1–3 cm wide. The finished joint is flush with both inside and outside with the pipe wall. The jointing space is filled with cement mortar mixed sufficiently dry to remain in position when forced with a trowel or rammer.

External Flush Joint – This joint is suitable for pipes which are too small for jointing from inside. Great care shall be taken in handling to ensure that the projecting ends are not damaged, as no repairs can be readily effected from inside the pipe. Details of the joint are shown in Figure 11.18 below.

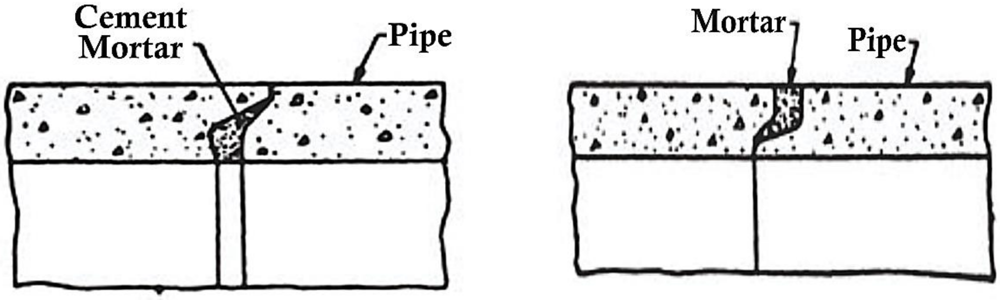


Figure 11.18: Internal and External Flush Joint

B. Flexible Joints

The water seal is effected because of compact pressure between the sealing rubber ring (or similar material) and the pipe surface. These are mainly two types.
Roll on Joint – A rubber ring (circular in cross-section) is placed at or near the end of the spigot and rolls along it as the spigot enters the socket.

Confined Gasket – Rubber ring of circular cross-section is held in the groove formed on the spigot. Sometimes, the cross-section is in the shape of a lip. The lips are opened due to water pressure which ensures water seal. For assembly of this joint, a lubricant has to be applied to the sliding surfaces. The lubricant washes off when the pipe is in service. Figure 11.19 shows confined O-ring joint and roll on joint.

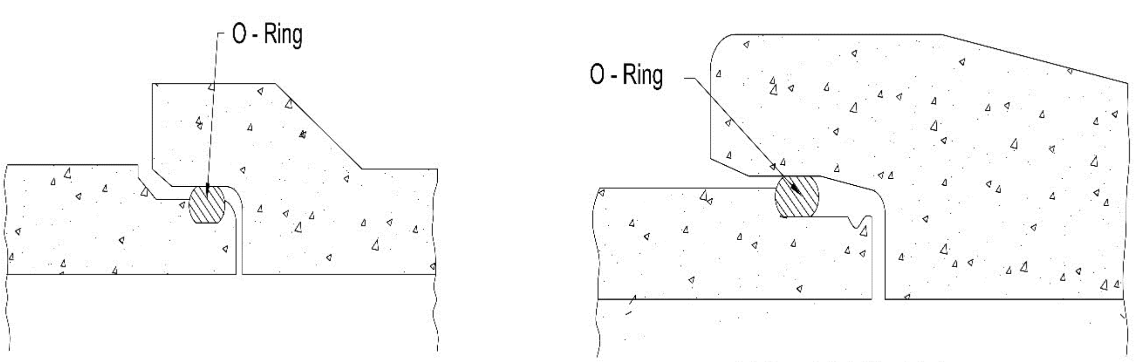


Figure 11.19: Confined O-ring joint and Roll on Joint

All pipelines should be tested before being brought into service. When testing the pipeline hydraulically, the line shall be kept filled completely with water for a week. The pressure shall then be increased gradually to full test pressure as indicated above at testing of the pipeline section under cast iron pipe and maintained at this pressure during the period of test with the permissible allowance indicated therein.

11.7.3 Advantages and Disadvantages

The advantages of pipe are:

- Good corrosion resistance,
- Widespread availability,
- High strength,
- Good load supporting capacity.

The disadvantages of pipe are:

- Require careful installation to avoid cracking,
- Heavy,
- Susceptible to attack from aggressive soils,
- Poor adaptability in installation,
- Making house connection is difficult as it is difficult to fix in ferrule which may cause leakage and weak connection.

11.8 Prestressed Concrete Pipes (PSC)

11.8.1 General

While reinforced cement concrete (RCC) pipes can cater to the needs where pressures are up to 6 kg/cm^2 and CI and steel pipes cater to the needs of higher pressures around 24 kg/cm^2, the prestressed concrete (PSC) pipes cater to intermediate pressure range, where RCC pipes would not be suitable.

The strength of a PSC pipe is achieved by helically winding high tensile steel wire under tension around a concrete core thereby putting the core into compression. When the pipe is pressurised, the stresses induced relieve the compressive stress, but they are not sufficient to subject the core to tensile stresses. The prestress wire is protected against corrosion by a surround of cementitious cover coat giving at least 18 mm thick cover. The PSC pipes are suited for water supply mains where pressures in the range of 6 kg/cm^2 to 20 kg/cm^2 are encountered.

Two types of PSC pipes are in use:

(i) **Cylinder-type:** Consists of a concrete lined steel cylinder monolith with other parts to be joint to cylinder. Steel joint rings welded to its ends wrapped with a helix of highly stressed wire and coated with dense cement mortar or concrete. Prestressed concrete cylinder pipe has the following two general types of construction:

   a. a steel cylinder lined with a concrete core; or
   b. a steel cylinder embedded in a concrete core.

   In either type of construction, manufacturing begins with a full-length welded steel cylinder. Joint rings are attached to each end and the cylinder is hydrostatically tested to ensure water tightness. A concrete core with a minimum thickness of one-sixteenth times the pipe diameter, or as per IS 784-2019, is placed either by the centrifugal process, radial compaction, or by vertical casting. After the core is cured, the pipe is helically wrapped with high strength, hard-drawn wire using a stress of 75 per cent of the minimum specified tensile strength. The wrapping stress is calculated on the basis of 75% the value of minimum ultimate tensile strength as per IS 1785: 1983, Reaffirmed 2018. The wire spacing is accurately controlled to produce a predetermined residual compression in the concrete core. The wire is embedded in a thick cement slurry and coated with a dense mortar that is rich in cement content.

The joints between PSC (cylinder-type) pipe will be site welded joint/rubber ring joint.

Size Range**:** Effective length of cylindrical pipe shall be up to 7.0 m. However, for pipes of diameter up to and including 300 mm, the effective length shall not exceed 3.0 m. Nominal internal diameter for cylindrical pipes is varying from 200 mm to 2500 mm (IS 784: 2019).

The technology for manufacture of these pipes is now available with Indian manufacturers.

(ii) **Non-cylinder type:** Consists of a concrete core which is pre-compressed both in longitudinal and circumferential directions by a highly stressed wire. The wire wrapping is protected by a coat of cement mortar or concrete.

Physical behaviour of PSC pipes under internal and external load is superior to RCC pipes. The PSC pipe wall is always in a state of compression which is the most favourable factor for permeability. These pipes can resist high external loads. The protective cover of cement and mortar which covers the tensioned wire wrapping by its ability to create and maintain alkaline environment around the steel inhibits corrosion. PSC pipes are joined with flexible rubber rings.

The deflection possible during laying of main is relatively small and the pipes cannot be cut to size to close gaps in the pipeline. Special closure units (consisting of a short double spigot piece and a plain-ended concrete lined/anti-corrosive food grade paint steel tube with a follower ring assembled at each end) are manufactured for this purpose. The closure unit (minimum length 1.27 m) must be ordered specially to the exact length.

Specials such as bends, bevel pipes, flanged tees, tapers, and adapters to flange the couplings are generally fabricated as mild steel fittings lined and coated with concrete/anti-corrosive food grade paint. It is worthwhile when designing the pipeline to make provision for as many branches as are likely to be required in the future and then to install sluice valves or blank flanges on these branches. It is possible to make connections to the installed pipeline by emptying, breaking out, and using a special closure unit.

11.8.2 Laying and Jointing

PSC non-cylinder pressure pipes are provided with flexible joints; the joints being made by the use of rubber gasket. They have socket spigot ends to suit the rubber ring joint. The rubber gasket is intended to keep the joint watertight under all normal conditions of service including expansion, contraction, and normal earth settlement. The quality of rubber used for the gasket should be waterproof, flexible, and should have a low permanent set. Figure 11.20 shows PSC (Non-Cylinder Type) Pipe Confined Joint Details and Figure 11.21 shows PSC (Non-Cylinder Type) Pipe Roll on Joint Details.

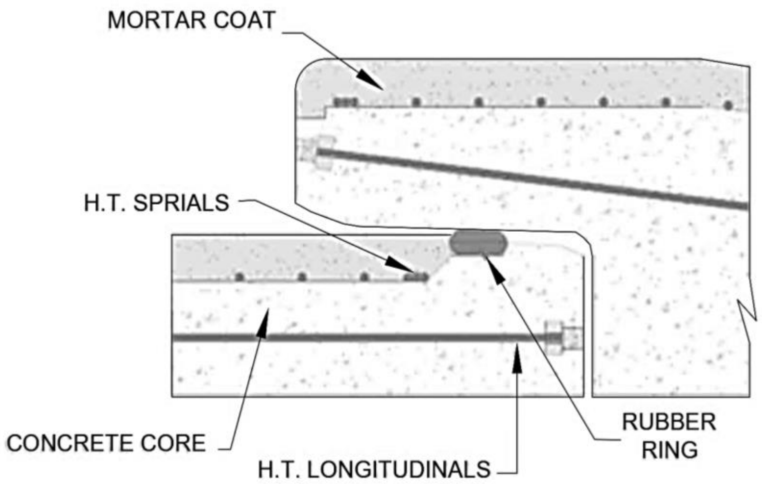


Figure 11.20: PSC (Non-Cylinder Type) Pipe Confined Joint Details

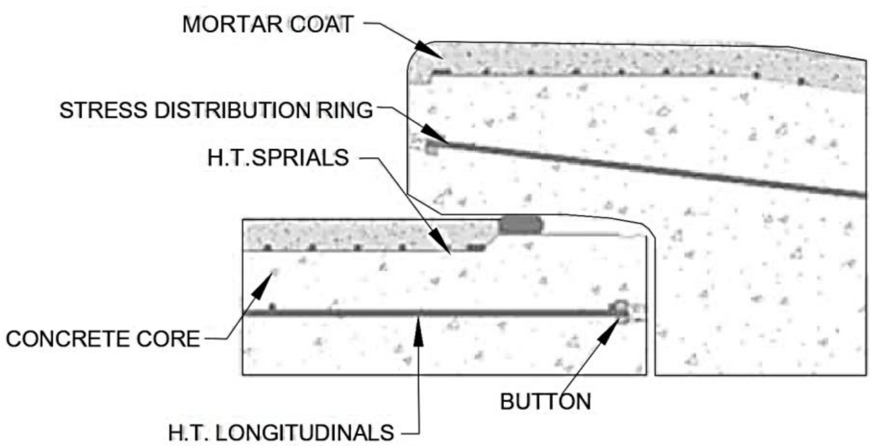


Figure 11.21: PSC (Non-Cylinder Type) Pipe Roll on Joint Details

Unless otherwise specified, joints between PSC cylinder pipes shall be of spigot and socket-type with rubber ring or with steel joint rings embedded at ends for site welding. In case of pipes for culverts, joints may be spigot and socket, roll on gasket joint, confined gasket joint or flush joint. The rubber ring joint design shall take into consideration the tolerance for rubber cord, tolerance for socket and spigot diameters, allowable deflection at joint and permanent set in the rubber ring. Figure 11.22 shows PCCP Pipe Sliding Overlap Welded Joint and Figure 11.23 PCCP Pipe Confined Rubber Ring Joint.

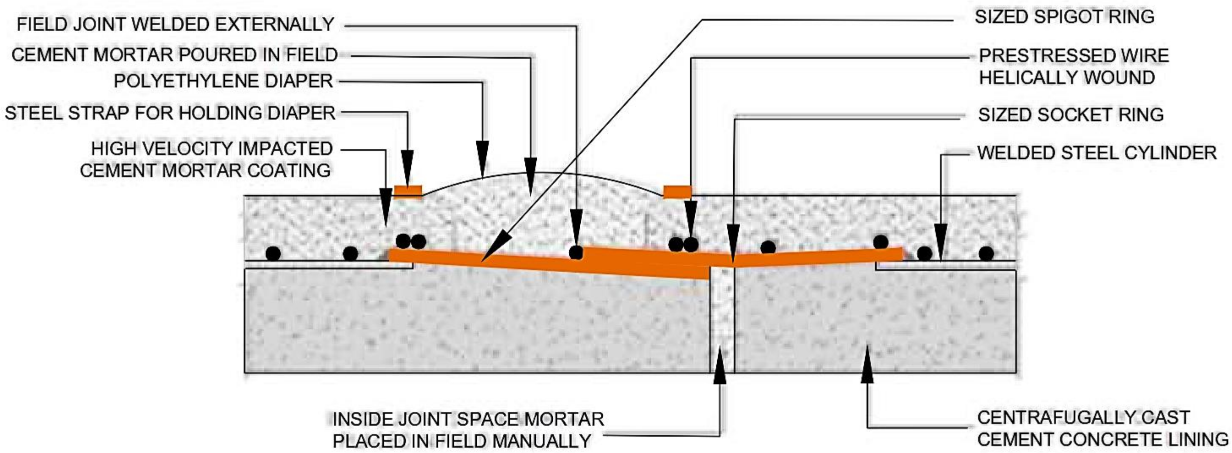


Figure 11.22: PCCP Pipe Sliding Overlap Welded Joint

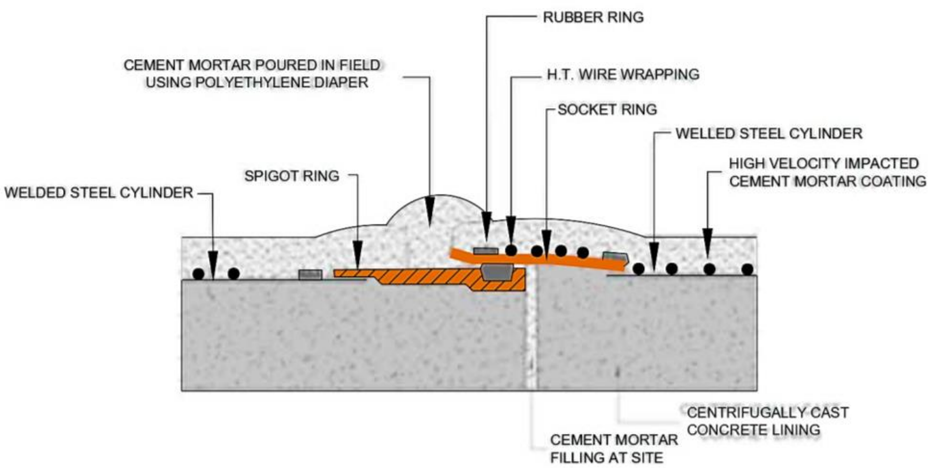


Figure 11.23: PCCP Pipe Confined Rubber Ring Joint

The sealing rings shall be of such size that when jointed, it shall provide a positive seal within the recommended range of maximum joint deflection not more than two splices in each ring shall be permitted.

The steel for fabricated steel plate specials, is cut, shaped, and welded so that the finished special has the required shape and internal dimension. Adjacent segments are joined by butt welding. Before lining and coating, the welding of specials shall be tested by use of hot oil or dye penetrant according to IS 3658: 1999, Reaffirmed 2020, and defects, if any, shall be rectified. The steel plate thickness for specials shall be as given in IS 1916. In dye penetrant inspection, a whitewash is applied over the weld on one side of the cylinder, on other side when coloured paraffin or similar product is applied over the weld, no coloured spot shall appear on the whitewash before four hrs. If any coloured spots appear before four hours, weld shall be repaired and retested.

The special shall be jointed to the pipe by same rubber ring joint as for pipes.

11.8.3 Testing of Pipelines

The pressure testing of PSC cylinder and non-cylinder pipes is the same as mentioned above at testing of the pipeline section under cast iron pipe. However, the quantity of water added in order to re-establish the test pressure should not exceed three litres (instead of 0.1 litres) per mm diameter, per km per 24 hours per 30 m head for non-absorbent pipes as per IS 783:1985, Reaffirmed 2022. The field test pressure shall be maintained for one hour. If the visual inspection indicates that there is no leakage, the test can be passed.

11.8.4 Advantages and Disadvantages

The advantages of pipe are:

- Good corrosion resistance,
- Widespread availability,
- High strength,
- Good load supporting capacity,
- Suitable for higher diameter pipeline (more than 1200 mm),
- Suitable for use in corrosive environment.

The disadvantages of pipe are:

- Require careful installation to avoid cracking,
- Heavy,
- Susceptible to attack from aggressive soils,
- Poor adaptability in installation,
- Difficult to repair and additional machine ends (sockets, spigots, barrels, etc.) need to be kept in inventory.

11.9 Bar/Wire Wrapped Steel Cylinder Pipes with Mortar Lining and Coating

11.9.1 General

Bar/Wire wrapped steel cylinder pipes with mortar lining and coating are available in diameters of 250 mm to 1900 mm and higher diameter pipes can be designed for working pressures up to 25 kgs/cm^2. Effective length of pipes shall be 4 m to 8 m. Longer length pipes can also be custom made. The manufacturer shall declare the length of pipe for any given design and the tolerance shall be applicable to that.

Manufacture of bar/wire wrapped steel cylinder pipes with mortar lining and coating begins with fabrication of a thin steel pipe cylinder. Thicker steel joint rings are welded at both ends. Each steel cylinder is hydrostatically tested. A cement mortar lining is placed by centrifugal process inside the cylinder. After the lining is cured by steam or water, mild steel rod is wrapped on the cylinder using moderate tension in the bar. The wrapping is to be done under controlled tension ensuring intimate contact with the cylinder. The cylinder and bar wrapping are covered with a cement slurry and a dense mortar coating that is rich in cement. The coating is cured by steam or water.

A welded steel sheet cylinder, which may be with steel socket and spigot rings welded to its ends for rubber ring joints or with steel rings welded to its ends for welded joints, lined with cement mortar centrifugally applied within the steel cylinder and spigot ring, with reinforcement consisting of continuous steel bar/wire helically wound around the outside of the cylinder and securely fastened by welding to the steel socket and spigot/joint rings, and subsequently coated with dense cement mortar covering the steel cylinder and bar/wires except for necessarily exposed socket and spigot joints rings. Figure 11.24 shows typical longitudinal section.

The wall thickness shall not be less than the design thickness by more than 5 per cent or 5 mm whichever is greater. The manufacturer shall declare the wall thickness for any given design. Tolerance on length of pipe shall be +2.5 per cent and -1 per cent of the specified length. The cement mortar coating shall provide a minimum cover of 19 mm over the bar/wire reinforcement or 25 mm over the cylinder, whichever is greater (as per IS 15155:2020).

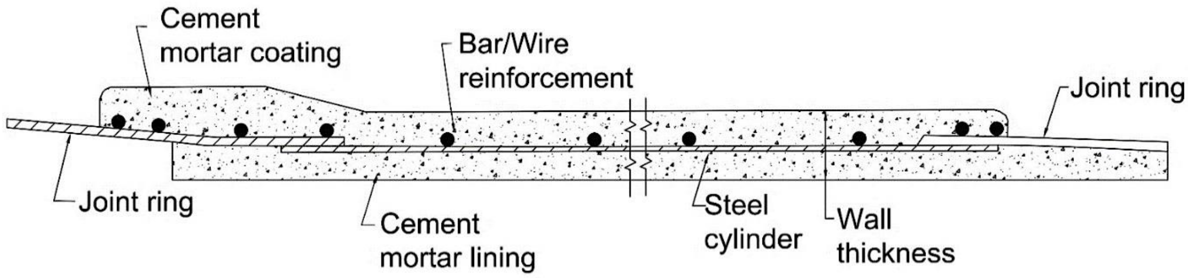


Figure 11.24: Typical Longitudinal Section

The purchaser may specify the application of an external or internal bituminous epoxy or other approved coating to be applied. When the pipes are to be used for carrying potable water, the inside lining shall not contain any constituents soluble in such water or any ingredient which could impart any taste or odour to the potable water.

11.9.2 Laying and Jointing

The standard joint consists of steel joint rings and a continuous solid rubber ring gasket. The field joint can be over lapping/sliding, butt welded or with confined rubber ring as per the client's requirement. In the case of welded and rubber joints, the exterior joint recess is normally grouted, and the internal joint space may or may not be pointed with mortar. These pipes can be laid in black cotton soil with additional precautions in bedding.

11.9.3 Testing of Pipelines

The pressure testing of BWSC pipes is the same as mentioned above at testing of the pipeline section under cast iron pipe. However, the quantity of water added in order to re-establish the test pressure should not exceed three litres (instead of 0.1 litres) per mm dia. per km per 24 hours per 30 m head as per the (IS 783:1985, Reaffirmed 2022).

The field test pressure shall be maintained for one hour. If the visual inspection indicates that there is no leakage, the test can be passed.

11.9.4 Advantages and Disadvantages

The advantages of pipe are:

- Pipes are having semi-rigid or semi-metallic properties and resistant to impact.
- Stiffer than conventional steel pipes.
- Corrugated surface increases its structural stability.

The disadvantages of the pipe are:

- Heavier than pipes like DI, HDPE, PVC, GRP pipes which makes it difficult to handle.
- Rough handling may damage the outer coat or inner lining.

11.10 Plastic Pipes

Plastic pipes are produced by extrusion process followed by calibration to ensure maintenance of accurate internal diameter with smooth internal bores. These pipes generally come in lengths of 6 metres. A wide range of injection moulded fittings, including tees, elbows, reducers, caps, pipe saddles, inserts and threaded adapters for pipe sizes up to 200 mm are available.

In house installations, plastic pipes cannot be used for electrical earthing being a non-conductive material. In colder climates, plastic pipes cannot be softened by conventional and electric equipment. Where pumps are used with plastics pipes, starting, and stopping are the occasions when damage may occur. The water hammer causes compression of the water in the pipe and consequently results in stretching of the pipe and where necessary, pressure relief devices should be included in the pipelines. Accurate records of laying of plastic pipes are very essential as they cannot be located by conventional electronic pipe locators.

In order to take care of the possible deteriorating effect by direct sunlight, it should be prevented by direct exposure to sunlight. For carefully executed installations using properly manufactured plastic pipes, no taste and odour problems should normally be encountered while in operation. The quality of plastic pipes should also be checked for its suitability from bacteriological point of view.

11.10.1 PVC Pipes

11.10.1.1 General

The PVC pipes are much lighter than conventional pipe materials. Because of their lightweight, PVC pipes are easy to handle, transport, and install. Solvent cementing technique for jointing PVC pipe lengths is cheaper, more efficient, and far simpler. PVC pipes do not become pitted or tuberculated and are unaffected by fungi and bacteria and are resistant to a wide range of chemicals. They are immune to galvanic and electrolytic attack, a problem frequently encountered in metal pipes, especially when burned in corrosive soils or near brackish waters. PVC pipes have elastic properties and their resistance to deformation resulting from earth movements is superior compared to conventional pipe materials especially AC. Thermal conductivity of PVC is very low compared to metals. Consequently, water transported in these pipes remains at a more uniform temperature.

Rigid PVC pipes weigh only one-fifth of conventional steel pipes of comparable sizes. PVC pipes are available in sizes of outer dia. 20, 25, 32, 50, 63, 75, 90, 110, 140, 160, 250, 290, and 315 mm at working pressures of 2, 5 ,4, 6, 10 Kg/cm^2 as per IS 4985 – 1988, Reaffirmed 2015. The wall of the plain pipe shall not transmit more than 0.2 per cent of the visible light falling on it when tested in accordance with IS 12235 (Part 1-19): 2004, Reaffirmed 2019.

Since deterioration and decomposition of plastics are accelerated by ultraviolet light and frequent changes in temperature which are particularly severe in India, it is not advisable to use PVC pipes above ground. The deterioration starts with discolouration, surface cracking, and ultimately ends with brittleness, and the life of the pipe may be reduced to 15–20 years.

Because of their light weight, there may be a tendency for the PVC pipes to be thrown much more than their metal counterparts. This should be discouraged, and reasonable care should be taken in handling and storing to prevent damage to the pipes. Under no circumstances should the pipes be dragged along the ground. Pipes should be given adequate support at all times. These pipes should not be stacked in large piles, especially under warm temperature conditions, as the bottom pipes may be structurally distorted thus giving rise to difficulty in pipe alignment and jointing. For temporary storage in the field, where racks are not provided, care should be taken that the ground is level, and free from loose stones. Pipes stored thus should not exceed three layers and should be so stacked as to prevent movement. It is also recommended not to store one pipe inside another. It is advisable to follow the practices mentioned as per IS 7634 (Part 1): 1975, Reaffirmed 2017 and IS 7634 (Part 3):2003, Reaffirmed 2018.

11.10.1.2 Laying and Jointing

A. Laying

The trench bottom should be carefully examined for the presence of hard objects such as flints, rocks, projections, or tree roots. In uniform, relatively soft fine-grained soils found to be free of such objects and where the trench bottom can readily be brought to an even finish providing a uniform support for the pipes over their lengths, the pipes may normally be laid directly on the trench bottom. In other cases, the trench should be cut correspondingly deeper and the pipes laid on a prepared under-bedding, which may be drawn from the excavated material if suitable.

The trench bed must be free from any rock projections. The trench bottom, where it is rocky and uneven, a layer of sand or alluvial earth equal to one-third diameter of the pipe or 100 mm, whichever is less, should be provided under the pipes.

As a rule, trenching should not be carried out too far ahead of pipe laying. The trench should be as narrow as practicable. This may be kept from 0.30 m over the outside diameter of pipe and depth may be kept at 0.60–1.0 m depending upon traffic conditions. Pipe lengths are placed end to end along the trench. The glued spigot and socket jointing technique as mentioned later is adopted. The jointed lengths are then lowered in the trench and when sufficient length has been laid, the trench is filled.

In laying, long lengths of pipe, prefabricated double socketed connections are frequently used to join successive pipe lengths of either the same or one size different. The socket in this case must be formed over a steel mandrel. A short length of pipe is flared at both ends and used as the socket connection. The mandrel used is sized such that the internal dia. of the flared socket matches the outer dia. of the spigot to be connected.

If trucks, lorries, or other heavy traffic will pass across the pipeline, concrete tiles 600 × 600 mm of suitable thickness and reinforcement should be laid about 2.0 m above the pipe to distribute the load. If the pipeline crosses a river, the pipe should be buried at least 2.0 m below bed level to protect the pipe. Individual pedestal approach may also be followed in case of long stretch of the river. The pipeline may also be laid down attached with the bridge piers across the river.

For bending, the cleaned pipe is filled with sand and compacted by tapping with wooden stick and pipe ends plugged. The pipe section is heated with flame and the portion bent as required. The bend is then cooled with water, the plug removed, the sand poured out and the pipe (bend) cooled again. Heating in hot air over hot oil bath, hot gas or other heating devices are also practised. Joints may be heat-welded, flamed, with rubber gaskets, or made with solvent cement. Threaded joints are also feasible, but are not recommended.

11.10.1.3 Jointing

Socket and spigot joint is usually preferred for all PVC pipes up to 150 mm in dia. The socket length should at least be one and a half times the outer dia. for sizes up to 100 mm dia. and equal to the outer dia. for larger sizes.

Jointing of PVC pipes can be made in following ways:

(i) Solvent cement
(ii) Rubber ring joint
(iii) Flanged joint
(iv) Threaded joint

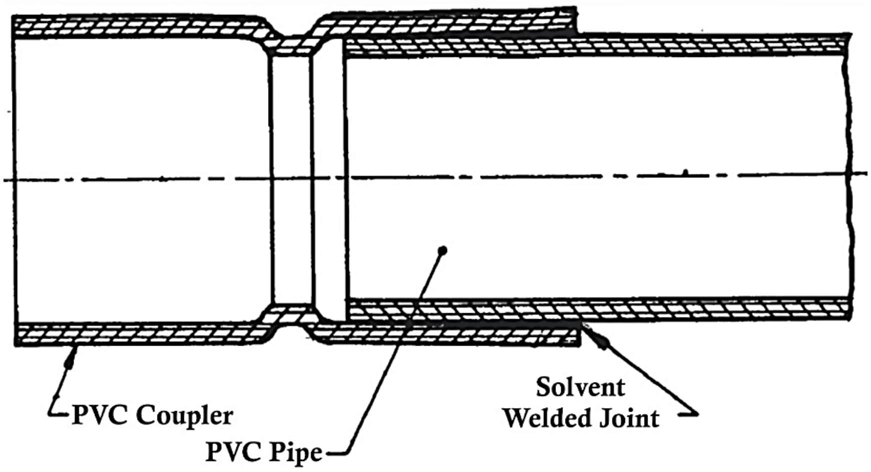


Figure 11.25: PVC Solvent Welded Joint

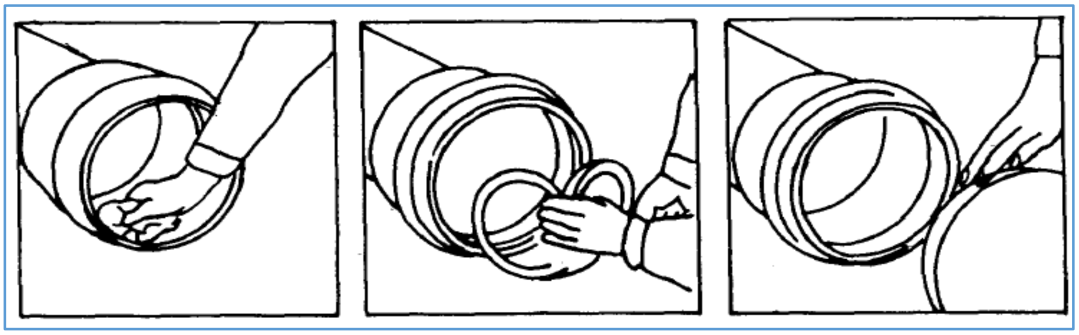


Figure 11.26: Sealing Ring Joint Assembly

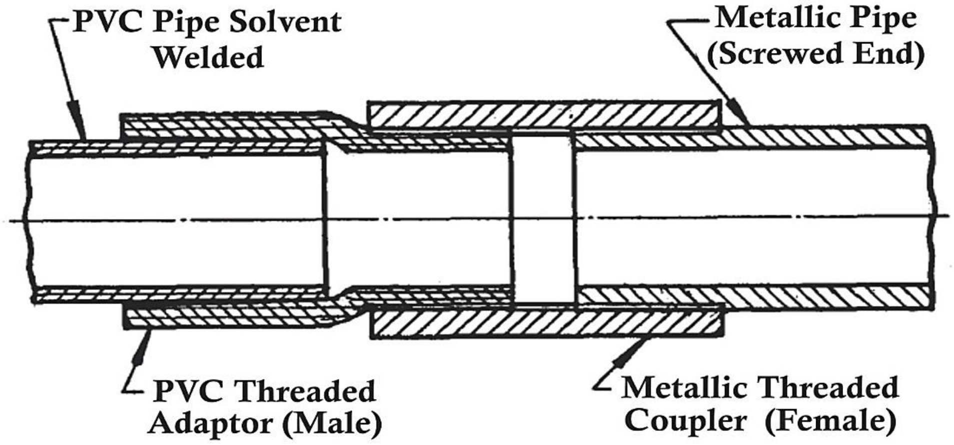


Figure 11.27: Threaded Joints with PVC

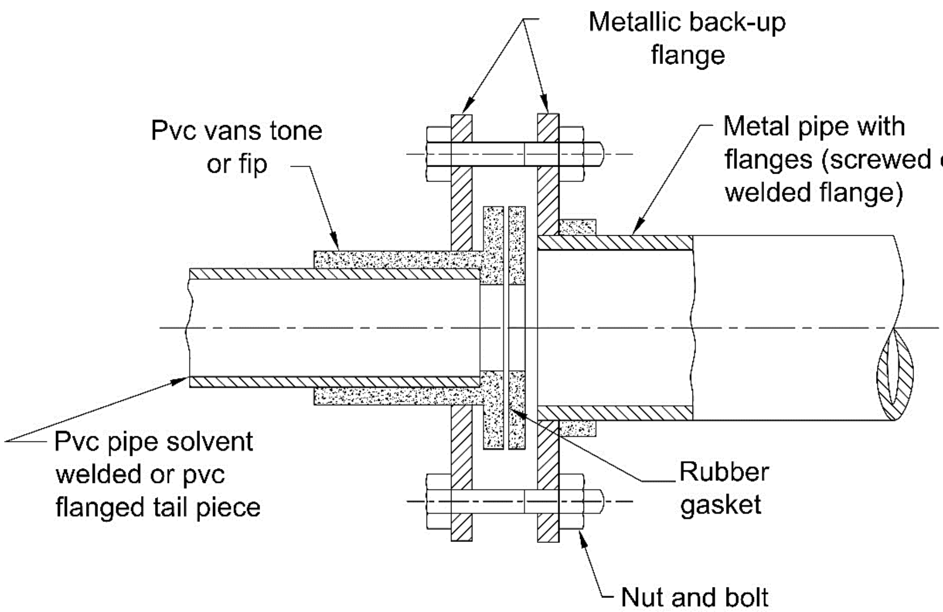


Figure 11.28: Flanged Joints with PVC

The above Figure 11.25 shows PVC solvent welded joint, Figure 11.26 shows sealing ring joint assembly, Figure 11.27 shows threaded joints with PVC, and Figure 11.28 shows flanged joints with PVC.

For pipe installation, solvent gluing is preferable to welding. The glued spigot socket connection has greater strength than can ever be achieved by welding. The surfaces to be glued are thoroughly scoured with dry cloth and preferably chamfered to 30°. If the pipes have become heavily contaminated by grease or oil, methylene cement is evenly applied with a brush to the outside surface of the spigot on one pipe, and to the inside of the socket on the other. The spigot is then inserted immediately in the socket up to the shoulder and thereafter, a quarter (90°) turn is given to evenly distribute the cement over the treated surface. The excess cement which is pushed out of the socket must be removed at once with a clean cloth. Jointing must be carried out in minimum possible time; time of making complete joint not being more than one minute. Joints should not be disturbed for at least five minutes. Half strength is attained in 30 minutes and full in 24 hours. Gluing should be avoided in rainy or foggy weather, as the colour of glue will turn cloudy and milky as a result of water contamination.

Normally, PVC pipes should not be threaded. For the connections of PVC pipes to metal pipes, a piece of a special thick wall PVC connecting tube threaded at one end is used. The other end is connected to the normal PVC pipe by means of a glued spigot and socket joint. Before installation, the condition of the threads should be carefully examined for cracks and impurities. Glue can be used for making the joints leak proof. Yarn and other materials generally used with metal pipe and fittings should not be used. Generally, it is advisable to use PVC as the spigot portion of the joint.

For further details on laying and jointing of PVC pipes, reference may be made to IS 4985 -2000, Reaffirmed 2015, IS 7634 (Part 1):1975 Reaffirmed 2017, IS 7634 (Part 2): 2012, Reaffirmed 2017, IS 7634 (Part 3):2003, Reaffirmed 2018.

Testing of pipelines

The pressure testing method, which is commonly in use, is filling the pipe with water, taking care to evacuate any entrapped air and slowly raising the system to appropriate test pressure. The pressure testing may be followed as same as mentioned above at testing of the pipeline section under cast iron pipe.

After the specified test time has elapsed, usually one hour, a measured quantity of water is pumped into the line to bring it to the original test pressure if there has been loss of pressure during the test. The pipe shall be tested as per para. 11.1 of IS 4985 (2021).

Advantage and Disadvantage

The advantages of pipe are:

- Resistance to corrosion,
- Lightweight,
- Toughness,
- Rigidity,
- Ease of laying, jointing, and maintenance,
- Ease of fabrication.

The disadvantages of pipe are:

- Deteriorating effect by direct sunlight,
- Water hammer causes stretching of the pipe,
- Non-conductive material.

11.10.2 Unplasticized Polyvinyl Chloride (UPVC) Pipes

11.10.2.1 General

The UPVC pipe is a plastic pipe made of polyvinyl chloride (PVC) resin and containing no plasticiser. Plastics belong to the group of newer pipe materials. With the development of chemical industry technology, it is now able to produce non-toxic grade pipes, so it has the function of usually polyvinyl chloride, and it has added some excellent functions, specifically its corrosion resistance and softness, so it is especially suitable for water supply networks. UPVC pipes are highly economical in comparison to pipes made from other materials. Plastic pipes offer high corrosion resistance to aggressive chemical media. Moreover, due to very smooth surfaces, the pipes are not prone to crust formation on the internal surface, which can have a detrimental effect on the water carrying capacity of the pipe. The pipes shall not have any detrimental effect on the composition of water flowing through them.

Pipes made from cast iron, fabricated steel and other materials are in use for a long time in various applications. Pipes for supplying drinking water are mostly made of polyethylene (PE) or polyvinyl chloride (PVC). Unplasticized PVC pipes are greatly used for transportation of water. It is an extruded product from a blend of polymer resin and various additives.

11.10.2.2 Laying and Jointing

A. Laying

Prolonged exposure of the pipes to sunlight must be avoided. Pipes must be protected from ultraviolet light (sunlight), which would otherwise cause discolouration and can reduce the impact strength of the pipe.

The depth of the trench shall be minimum 1.0 m. The width of the trench should be uniform throughout the length and greater than the outside diameter of the pipe by 300 mm on either side of the pipe. It should be ensured that the pipe have been laid along the central line of the trench. The trench bottom shall be constructed to provide a firm, stable, and uniform support for the full length of the pipeline. There should not be any sharp objects on the trench surface while laying the pipeline. Any large rocks, hard pan, or stones larger than 20 mm should be removed to permit a minimum bedding thickness of 100–150 mm under the pipe.

Place the pipe and fittings into the trench using ropes or by hand or mechanical means. The pipes should not be thrown into the trench or allow any part of the pipe to take an unrestrained fall onto the trench bottom.

Laying of the pipes in the trench after ensuring that bell holes have been provided for at the appropriate places in the bedding (pipes of diameter 110 mm or less, with no live load application, do not require bell holes in the trench bottom). The trenches should be refilled carefully after testing of the pipeline. The pipes should be laid with the spigots entered into the sockets in the same direction as the intended flow of water.

B. Jointing

Commonly used joints are (i) Solvent welded joints; (ii) Elastomeric sealing ring joints; (iii) Mechanical compression joints; (iv) Flanged joints; (v) Screwed or threaded joints; and (vi) Union coupled joints.

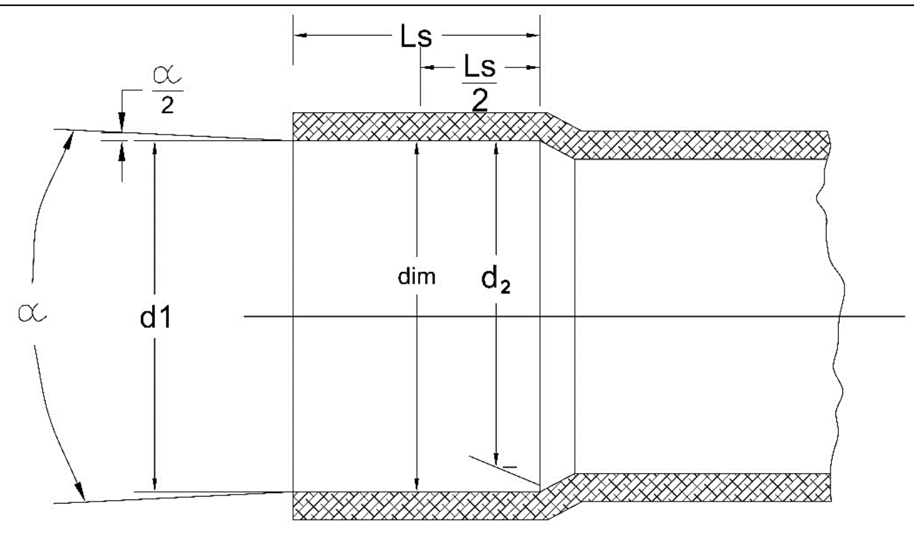
Sockets formed on the ends of the pipes shall be reasonably parallel to the axis of the pipe. The minimum length of any socket shall be given by the expression. Figure 11.29 shows socket dimensions for solvent cement joints and Figure 11.30 shows sockets for use with elastomeric sealing rings.

$$L_s = 0.5dn + 6mm \tag{11.3}$$

Where

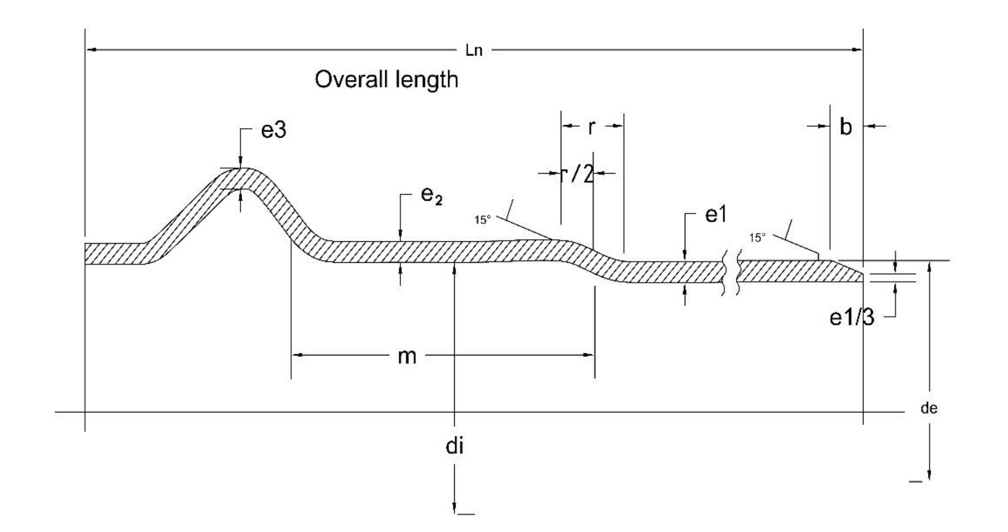
L = minimum socket length, and

dn = nominal outside diameter of the pipe.



Where:
d_{im} = diameter at mid-point of socket length;
d_1 = diameter at mouth;
d_2 = diameter at root;
L_s = socket length; and
α = total included angle of taper

Figure 11.29: Socket Dimensions for Solvent Cement Joints



Where:
m = minimum depth of engagement;
d_e = outer diameter at any point; and
d_i = inner diameter of socket, measured at mid-point of socket length, m

Figure 11.30: Sockets for use with Elastomeric Sealing Rings

The ends of the pipes meant for solvent cementing (both plain and bell ended) shall be cleanly cut and shall be reasonably square to the axis of the pipe or may be chamfered at the plain end. Pipes with plain end(s) to be used for elastomeric sealing ring-type joints shall be chamfered at approximately 15° to the axis of the pipe. Approximately two-thirds of the full wall thickness shall be chamfered.

11.10.2.3 Testing of pipelines

Pressure testing method which is commonly in use is filling the pipe with water, taking care to evacuate any entrapped air, and slowly raising the system to appropriate test pressure. The pressure testing may be followed as mentioned in IS 4985.

11.10.2.4 Advantages and Disadvantages

The advantages of pipe are:

- Very lightweight,
- Easy to install,
- Good corrosion resistance and cheap in cost,
- Smooth surface reduces friction losses,
- Long pipe sections reduce leakage/infiltration potential,
- Flexible.

The disadvantages of pipe are:

- Susceptible to chemical attack especially solvents,
- Strength affected by sunlight,
- Not suitable for above ground installations,
- Require great care during laying,
- Susceptible to damages due to external pressure and blows in above ground level application.

11.10.3 Oriented Polyvinyl Chloride (OPVC) Pipes

11.10.3.1 General

Amorphous polymer of PVC in which the molecules are located randomly. However, under certain conditions of pressure, temperature, and speed by stretching the material, it is possible to orient the polymer molecules in the same direction. The result is a plastic with a layered structure known as oriented polyvinyl chloride pipes of highest orientation class 500, with homogeneous socket elastomeric sealing ring.

It is a new technology for manufacturing pipes, which involves process of controlling circumferential and axial orientation of molecular structure resulting in the formation of laminar structure of the material used in the pipe construction, commonly named as oriented polyvinyl chloride (OPVC). It is manufactured as per Indian Standard IS 16647: 2017 (Reaffirmed 2022) – Oriented Unplasticized Polyvinyl Chloride (PVC-O) Pipes for Water Supply –Specification. This standard specifies the requirements of oriented unplasticized polyvinyl chloride (PVC-O) pipes, for piping systems intended to be used underground, or above ground, but not exposed to direct sunlight, for water supply.

Oriented pipes made from a defined PVC-U compound and with a well-defined orientation level in circumferential and axial direction, shall be evaluated according to the procedures specified in ISO 16422/ISO 9080 on the basis of tests given in 9.1.1 (see also 5.6.3).

Testing of compound mix used in manufacturing of pipes for its conformity for minimum required strength requirements, which require testing time of 10,000 hrs., is not covered in the IS 16647: 2017 (Reaffirmed 2022) due to the limited testing facility available, at present, in the country. The manufacturers have to ensure this before supplying pipes.

The piping system is intended for the conveyance of water, including potable water, up to and including 45 °C. The pipes are manufactured for different sizes in India, i.e., 110 mm, 160 mm, 200 mm, 250 mm, 315 mm, and 400 mm with pressure ratings of PN 12.5, PN 16, PN 20, and PN 25. The pipes are recommended for water temperatures ranging from 1 °C to 45 °C. At temperatures higher than 27 °C and up to 45 °C, the strength of the pipe reduces and working pressure shall therefore be modified using the derating factor as obtained from the graph given in Annexure B of IS 16647:2017 (Reaffirmed 2022).

The material from which the pipe is produced shall consist substantially of unplasticized polyvinyl chloride to which may be added only those additives that are needed to facilitate the manufacture of the pipe and the production of sound and durable pipe of good surface finish, mechanical strength, and opacity under conditions of use. Store the pipes horizontally on a flat surface and place supports every 1.5 m to avoid the bending of the product. Do not stack pipes more than 1.5 m height, as this can damage lower pipes or even the upper pipes can fall. In case of prolonged sun exposure, protect pallets with an opaque material. White colour is preferable because it avoids overheating of the pipes.

11.10.3.2 Laying and Jointing

When checked without magnification glass, the internal and external surfaces of the pipe shall be smooth, clean, and free from scoring, cavities, and other surface defects. The ends of the pipe shall be either cut cleanly and reasonably square to the axis of the pipe or chamfered at the plain end at approximately 15° to the axis of the pipe. The pipes shall be supplied with the length not less than the declared nominal pipe length. It is recommended that the nominal pipe length to be supplied may be 6 m, 10 m, and 12 m. The pipes may be supplied in other lengths where so agreed upon between the manufacturer and the purchaser.

Before placing the pipe, a sand bed should be prepared (a fine granular material may be used instead of sand) with a thickness from 10 cm to 15 cm. The pipe should be well aligned and levelled. The trench shall be free of stones at the bottom and at the sides. Stones smaller than 10 - 20 mm are allowed, but it cannot be the main size of the ground particles. The pipe shall lie on the sand bed. Once the pipe is placed, chamberlain sides shall be filled with the selected material and compacted to achieve >95 per cent proctor normal (PN). The trench shall be filled with the selected material and compacted laterally until the upper part of the pipe is buried at least 30 cm. Minimum width of the trench based on nominal diameter of pipes to be laid and/or depth of trench is specified in IS 16647: 2017. As a rule of thumb, when there is no road traffic involved, the pipe's crown will be at a minimum depth of 0.6 m; with road traffic, the minimum depth is 1.0 m.

Assembly details are as under:

a) Remove the protection caps, if any.
b) Verify that the pipe is clean and in good condition. Paying attention to the sockets and spigot ends.
c) Check that the chamfer is correct and free of cracks.
d) Verify that the seal is in its place, clean and free of foreign materials (stones, sand, etc.).
e) Lubricate the chamfer of the spigot and the seal with joint lubricant.
f) Line up the pipe as much as possible, horizontally and vertically.
g) Insert only the chamfer edge of the socket, just to support the pipe but leaving the socket lip free.
h) In the case of pipes with a nominal diameter of ≤250 mm, a firm and dry push should be given to seize the momentum produced by the free movement in the lip of the socket and introduce it until the mark is hidden into the socket.
i) When installing diameters >250 mm, one should use mechanical means to introduce the pipe using materials such as wood, hoists, tackles, or slings.

11.10.3.3 Testing of pipelines

Testing shall be performed only after the pipeline has been properly filled, flushed, and purged of all air. The specified test pressure shall be applied by means of an approved pumping assembly connected to the pipe properly and to prevent pipe movement, the contractor shall have placed enough backfill prior to filling and testing of the pipe. If necessary, the test pressure shall be maintained by additional pumping for the specified time during which the system and all exposed pipe, fittings valves, and hydrants shall be carefully examined for leakage. All visible leaks shall be stopped. All defective elements shall be repaired or removed and replaced. The test shall be repeated until the test requirements have been met. Pressure testing method, which is commonly in use, is filling the pipe with water, taking care to evacuate any entrapped air and slowly raising the system to appropriate test pressure. The pressure testing may be followed as mentioned in IS 16647: 2017 (Reaffirmed 2022) - Oriented Unplasticized Polyvinyl Chloride (PVC-O) Pipes for Water Supply.

Resistance to hydrostatic pressure shall be verified using the induced stresses derived from the analysis of the test data in accordance with IS 16462: 2016 (Reaffirmed Year: 2021) Plastic Piping and Ducting Systems – Determination of the Long-Term Hydrostatic Strength of Thermoplastics Materials in Pipe Form by Extrapolation. For a period of 10 hrs. at 27 °C and 1000 hrs. at 27 °C, the 99.5 per cent LPL value shall be taken as the minimum stress level. The test shall be carried out not earlier than 24 hrs. after the pipes have been manufactured (IS 16462: 2016 (Reaffirmed Year: 2021) Plastic Piping and Ducting Systems- Determination of the Long-Term Hydrostatic Strength of Thermoplastics Materials in Pipe Form by Extrapolation).

11.10.3.4 Advantages & Disadvantages

The advantages of pipe are:

- Light in weight and easy to handle.
- The high flexibility of the pipes enables withstanding large deformations without suffering structural damages.
- PVC-O is immune to corrosion so it does not require any coating or special protection.

- These pipes can endure higher internal pressures than other PVC pipes.
- The lower celerity figure of the pipes virtually eliminates the possibility of breakages that can occur during the process of opening/closing valves or when starting pumping operations.

The disadvantages of pipe are:

- Susceptible to chemical attack especially solvents.
- As OPVC pipe reduces its strength due to direct sunlight, the pipes shall be stacked with proper covered storage and shall not be used above ground installations.
- For OPVC fittings, no BIS is available at present.
- OPVC pipes shall not be used in rocky strata without proper special bedding. At present, there is no BIS specification for rocky areas.

11.10.4 Chlorinated Polyvinyl Chloride (CPVC) Pipes

11.10.4.1 General

The material from which the pipe is produced shall consist substantially of chlorinated polyvinyl chloride (CPVC) to which may be added only those additives that are needed to facilitate the manufacture of the pipe and the production of sound and durable pipe of good surface finish, mechanical strength, and opacity under conditions of use.

The CPVC polymer from which the pipe compound is to be manufactured shall have chlorine content not less than 66.5 per cent. The CPVC pipe compounds containing additives such as modifiers, lubricants, fillers, etc., from which the pipes are to be manufactured, shall have a density between 1450 kg/m^3 and 1650 kg/m^3, when tested in accordance with IS 15778 2007, Reaffirmed 2022.

The outside diameter at any point shall be measured according to the method given in IS 12235 (Part 1): 2004, Reaffirmed 2019.

BIS IS 15778: 2007, reaffirmed 2022- class 7.1.1 mentions that the permissible variation between the outside diameter at any point (de) and the nominal diameter (da) of a pipe (also called tolerance on ovality) shall not exceed the greater of the following two values:

(a) 0.5 mm, and
(b) 0.012 d_n mm rounded off to the next higher 0.1 mm, where d_n is the nominal diameter of pipe in mm.

The wall of the plain pipe shall not transmit more than 0.1 per cent of the visible light falling on it when tested in accordance with IS 12235 (Part 3): 2004, Reaffirmed 2019. The ends of the pipes meant for solvent cementing shall be cleanly cut and shall be reasonably square to the axis of the pipe or maybe chamfered at the plain end.

11.10.4.2 Laying and Jointing

CPVC pipes of all sizes are packed in polyethylene packing rolls and both the ends of the packed rolls are sealed with air bubble film cap in order to provide protection during handling and transportation. Visually inspect pipe ends before making the joint. The use of a chamfering tool will help identify any cracks by catching on to them.

Pipe may be cut quickly and efficiently by several methods. Wheel-type plastic tubing cutters are preferred. Ratchet-type cutter or fine-tooth saw are other options. However, when using the ratchet cutter, be certain to score the exterior wall by rotating the cutter blade in circular motion around the pipe. Do this before applying significant downward pressure to finalise the cut. This step leads to a square cut. Cutting tubing as squarely as possible provides optimal bonding area within a joint.

When making a joint, apply a heavy, even coat of cement to the pipe end. Use the same applicator without additional cement to apply a thin coat inside the fitting socket. Do not allow excess cement to puddle in the fitting and pipe assembly. This could result in a weakening of the pipe wall and possible pipe failure when the system is pressurised.

When making a transition connection to metal threads, use a special transition fitting or CPVC male threaded adapter whenever possible. Do not over torque plastic threaded connections. Hand tight plus one-half turn should be adequate. Hang or strap CPVC systems loosely to allow for thermal expansion. Do not use metal straps with sharp edges that might damage the tubing.

Testing of Pipeline

When subjected to internal hydrostatic pressure test in accordance with the procedure given in IS 12235 (Part 8): 2004, Reaffirmed 2019, the pipe shall not fail during the prescribed test duration. The temperatures duration and hydrostatic (hoop) stress for the test shall conform as per IS 12235 (Part 8): 2004, Reaffirmed 2019, the test shall be carried out not earlier than 24 hrs. after the pipes have been manufactured.

11.10.4.3 Advantages and Disadvantages

The advantages of pipe are:

- Lightweight and easy for transportation;
- Requires fewer tools for installation and maintenance;
- Lack of plasticisers which discourages microbial growth;
- Corrosion and abrasion resistance;
- Reground into pellets and recycled; and
- Reduces heat loss due to lower thermal conductivity.

The disadvantages of pipe are:

- High thermal expansion coefficient.

11.10.5 Polyethylene (PE) Pipes

11.10.5.1 General

The IS 4984 (2016, Reaffirmed 2021) – PE pipes for water supply lays down the requirements for polyethylene (PE) pipes (mains and service pipes) intended for the conveyance of water for human consumption including raw water prior to treatment and also water for general purpose. This standard is applicable for the water supplies with a maximum operating pressure of 2.0 MPa.

As polyethylene pipes are designated by their minimum required strength (MRS), the earlier nomenclature of high-density polyethylene pipes has been renamed as polyethylene pipes and accordingly, the title of the standard has also been modified. Pipes shall be classified according to the grade of the raw material (resin) as PE 63, PE 80, PE 100. The pipe sizes are 16 mm to 2000 mm dia.

These pipes are not brittle, and as such, a hard fall at the time of loading and unloading, etc. may not do any harm to it. Polyethylene is a tough resilient material which may be handled easily. Abrasion resistance is much higher compared to metal or concrete pipes. However, because it is softer than metals, it is prone to damage by abrasion and by objects with a cutting edge. The pipes are highly resistant to notches and scratches and therefore widely used in trenchless installation like horizontal direction drilling (HDD) where damage possibility is very high, apart from joint integrity.

Polyethylene pipes incorporated with 2–2.5% of finely dispersed carbon black, have unlimited resistance against sunlight, and can deliver lifetime service above ground. HDPE materials have excellent resistance against strong acids, alkalis, salts, and most chemicals, which are commonly packed in HDPE drums and barrels. They do not undergo galvanic corrosion, hence, there is no need for any coating or cathodic protection. As there is no corrosion, the water quality in PE pipes is most secured as corrosion products do not get added to it.

11.10.6 High Density Polyethylene (HDPE) Pipes

11.10.6.1 General

HDPE pipe is a type of flexible plastic pipe used for water supply systems. It is made from the high density polyethylene which is suitable for high-pressure pipelines.

The material used for the manufacture of pipes should not constitute toxic hazard, should not support microbial growth, and should not give rise to unpleasant taste or odour, cloudiness, or discolouration of water. The percentage of antioxidant used shall not be more than 0.3 per cent by mass of finished resin.

Among the recent developments is the use of high density polyethylene pipes. These pipes are not brittle and as such a hard fall at the time of loading and unloading, etc., may not do any harm to it. HDPE pipes as per IS 4984: 2016 can be joined with detachable joints and can be detached at the time of shifting the pipeline from one place to another. Though for all practical purposes, HDPE pipes are rigid and tough. At the same time, they are resilient and conform to the topography of land when laid over ground or in trenches. They can withstand movement of heavy traffic. This would not cause damage to the pipes because of their flexural strength.

HDPE pipes are non-susceptible for tracing instruments because of its inertness. Copper or metallic conductor-extruded HDPE pipes have also been developed to increase the traceability of pipes (by instruments) when buried underground.

11.10.6.2 Laying and Jointing

A. Laying

The pipeline may be laid alongside of the trench and jointed there outside of it. Hence, the trench size is relatively small (no man entry trench is needed), which saves lot of civil work, installation time and cost. Thereafter, the jointed pipeline shall be lowered into the trench carefully without causing undue bending. The pipeline shall be laid inside the trench with a slack of up to 2 m/100 m of pipeline. The trench depth should be OD+300 mm.

Polyethylene (PE) pipe requires no special bed preparation for laying the pipe underground, sieved, excavated material is good enough, except that there shall be no sharp objects around the pipe. However, while laying in rocky areas, suitable bedding should be provided around the pipe and compacted, as per BIS specifications.

Polyethylene (PE) pipes are non-metallic, so once buried, metal detector-type locators are ineffective. To facilitate locating a buried PE pipe, metallic locating tapes or copper wires can be placed alongside the pipe. Locating tapes/wires are placed slightly above the crown of the above before the final backfill.

B. Jointing

Polyethylene (PE) pressure piping systems jointed by butt welding, electrofusion and flanges do not require external joint restraints or thrust block joint anchors. The HDPE pipe jointing with the use of electrofusion fittings jointing done with electrical resistance element incorporated in the socket of the fitting which, when connected to an appropriate power supply, melts and fuses the materials of the pipe and fitting together. Electrofusion joint can be made with the use of electrofusion machine, which should operate automatically only by scanning bar code on fittings. Since this jointing is done at outer diameter of pipe and jointing surface area being more, the joint can be strong and leak proof.

The BIS Standard IS 15927: PART 3: 2011 (Reaffirmed Year: 2021) is available for “Polyethylene Fittings for use with Polyethylene Pipes for the Supply of Gaseous Fuels - Part 3 Electro Fusion Fittings”. BIS Standard is not available for Electro Fusion Fittings for PE pipes for supply of water. However, EN ISO 12201-3/ISO 4427-3/BIS EN ISO 15494:2003 is available for such fittings. These fittings can be used in water supply line due to its advantages of fusion joint over other mechanical joints. The jointing can be faster, reliable, simple installation, homogenous joint which can be used in restricted trench space.

Commonly used joints are as follows:

(i) Fusion welding:
  a) Butt fusion welding; (Figure 11.31)
  b) Socket fusion welding; (Figure 11.32)
  c) Electrofusion welding; (Figure 11.33)
(ii) Insert-type joints; (Figure 11.34)
(iii) Compression fittings/push fit joints; (Figure 11.35)
(iv) Flanged joints; (Figure 11.36)
(v) Spigot and socket joints.

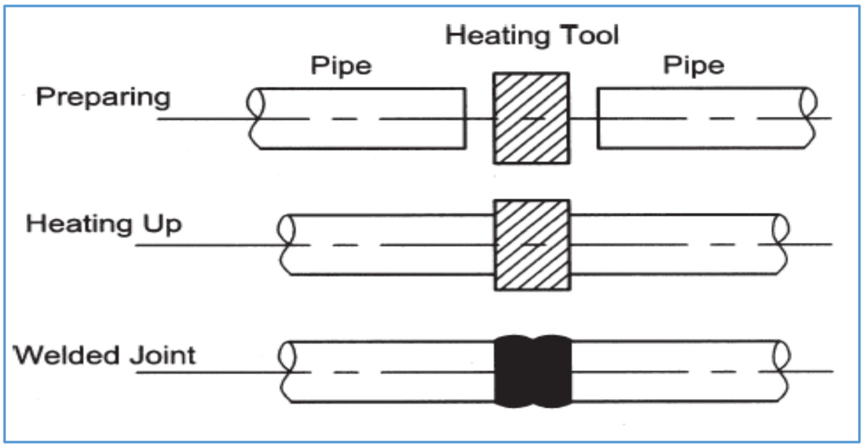


Figure 11.31: Butt Fusion Welding Procedure

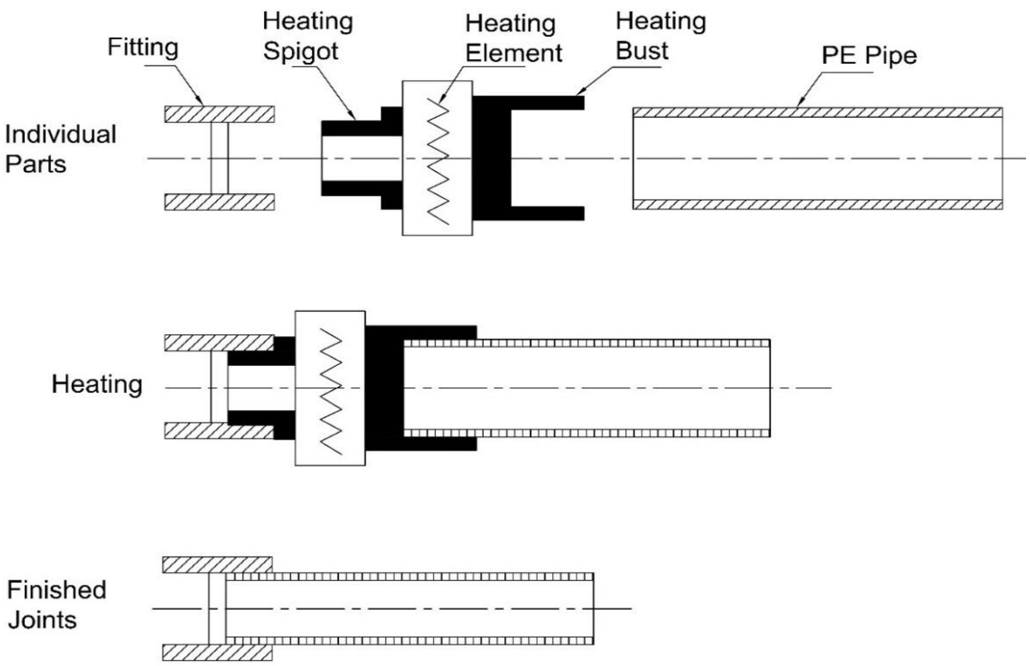


Figure 11.32: Socket Fusion Jointing Procedure

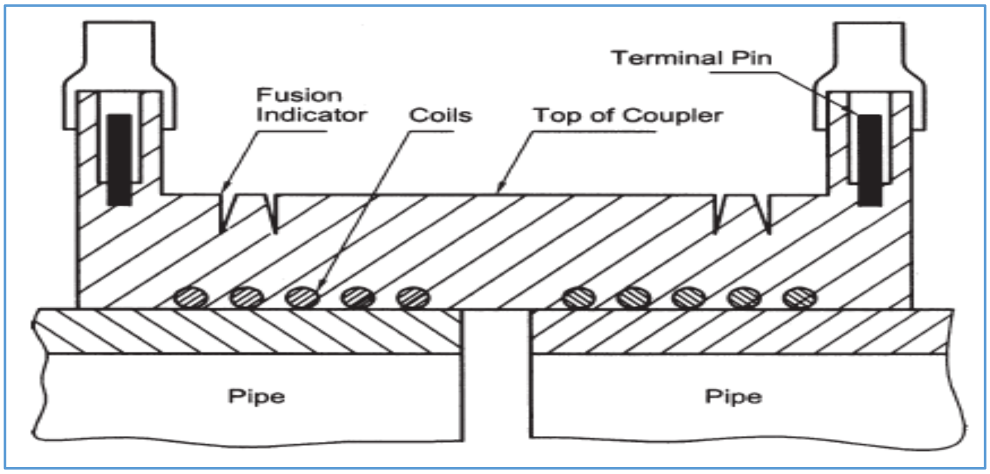


Figure 11.33: Electrofusion Process

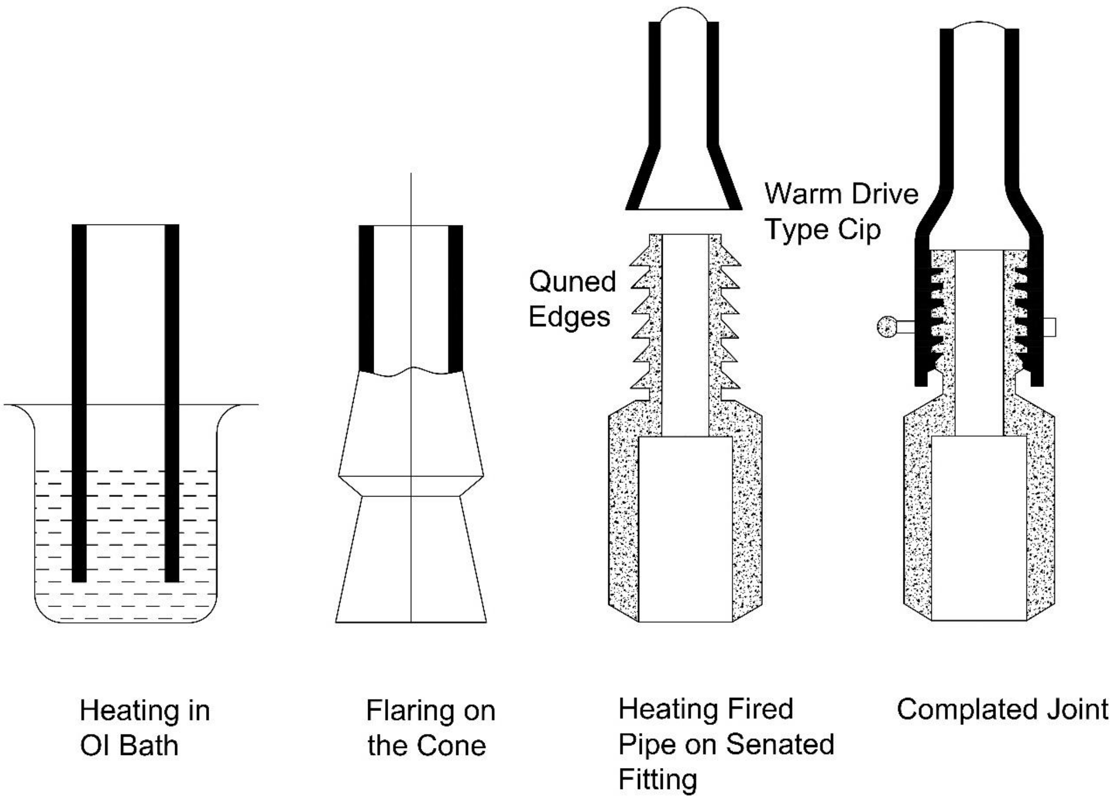


Figure 11.34: Insert-Type Joints

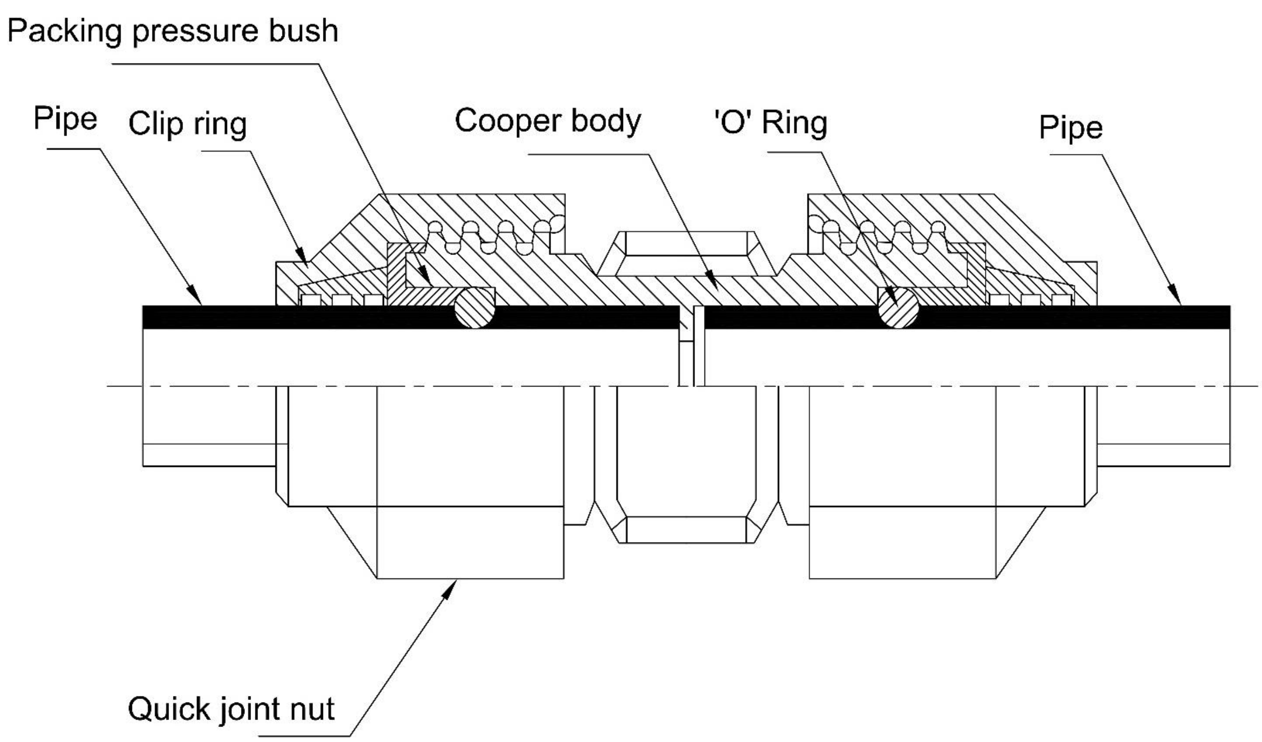


Figure 11.35: Polypropylene Compression Coupler Socket

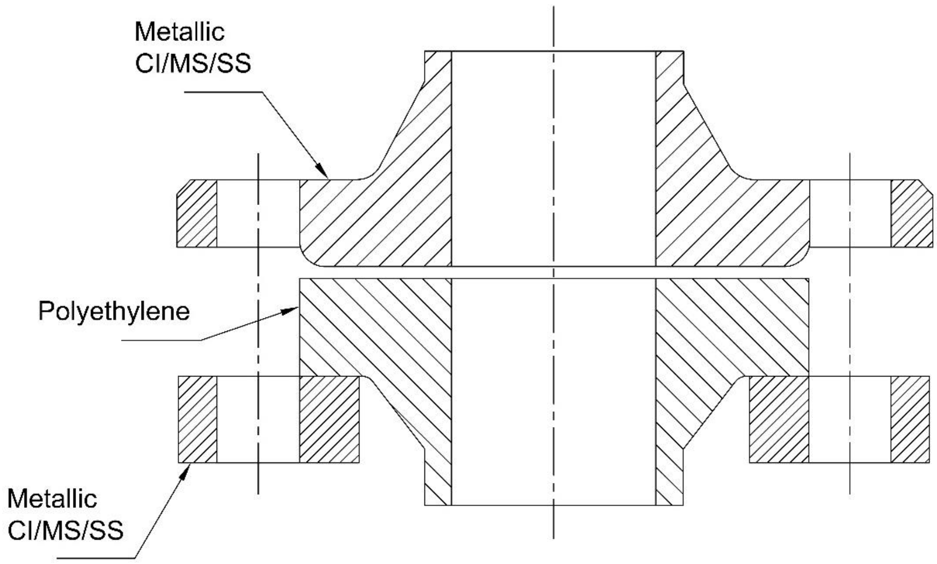


Figure 11.36: Typical Flanged Joint

PE Fittings

Polyethylene pipe fittings are covered under BIS code IS 8008:2022 – Injection Moulded/Machined Polyethylene Fittings for Water Supply-Specification and IS 8360:2022 – Fabricated Polyethylene Fittings for Water Supply-Specification, which may be referred to and used for connecting the pipes and other system appurtenances. These fittings can also be used for connecting to metallic valves (sluice, scour, and air), tanks, pipes, and other mechanical equipment (pumps, etc). However, where there is a likely hood of vibrations and turning torques in such connections, the fitting wall thickness shall be a minimum of one rating higher than the corresponding pipe.

11.10.6.3 Testing of pipelines

Pressure testing method, which is commonly in use, is filling the pipe with water, taking care to evacuate any entrapped air and slowly raising the system to appropriate test pressure. The pressure testing may be followed as mentioned in

(a) IS 7634: Part 2: 2012 (Reaffirmed Year: 2022) – Plastics Pipes Selection, Handling, Storage and Installation for Potable Water Supplies – Code of Practice Part 2 Laying and jointing of polyethylene (PE) pipes, and
(b) IS 4984: 2016 (Reaffirmed Year: 2021) – Polyethylene Pipes for Water Supply – Specification (Fifth Revision).

11.10.6.4 Advantages and Disadvantages

The advantages of pipe are:

- Lightweight,
- Easy to install,
- Good corrosion resistance,
- Smooth surface reduces friction losses,
- Long pipe sections reduce leakage/infiltration potential,
- Flexible.

The disadvantages of pipe are:

- Susceptible to chemical attack especially solvents,
- Strength affected by sunlight,
- Not suitable for over the ground installations.

11.10.7 Medium Density Polyethylene (MDPE) Pipes

11.10.7.1 General

Medium Density Polyethylene Pipes (MDPE), by definition, have a density between 0.926–0.940 gms/cc. A MDPE grade for pipes should have PE80 pressure classification for conveying potable water. The medium density polyethylene pipes are now being manufactured in India conforming to ISO specifications (ISO 4427 and BS 6730-1986) for carrying potable water. However, BIS is not available for these pipes. The MDPE pipes are being preferably used for consumer connection pipes as an alternative to GI pipes. The polyethylene material does not constitute toxic hazard and does not support any microbial growth. Further, it does not impart any taste, odour, or colour to the water.

MDPE pipes are colour-coded black with blue strips in sizes ranging from 20 mm to 110 mm diameter for pressure class of PN3.2, PN4, PN6, PN10, and PN16. The maximum admissible working pressures are worked out for temperature of 20^0 C (ISO4427: 2019). The pipes are supplied in coils and minimum coil diameter is about 18 times diameter of the pipe.

MDPE compression fittings made of PP, AABS, UPVC are also available in India for use with MDPE pipes. The materials used for the fittings are also suitable for conveying potable water like MDPE pipes. The jointing materials of fittings consists of thermoplastic resins of Polyethylene-type, NBR ‘O’ ring of nitrile and clamp of polypropylene, copolymer body, zinc plated steel reinforcing ring, nuts, and balls of special NBR gasket.

The MDPE pipes are lightweight, robust, and non-corrodible and hence, can be used as alternative material for consumer connections. Since the pipes are supplied in coils, there will be no joints under the roads and bends are avoided resulting in fast, simple, and efficient jointing.

MDPE is a popular material for urban water supply and is lightweight, strong, and flexible. MDPE pipes can be used for:

- Water distribution for town, rural, and irrigation projects,
- Cold water plumbing reticulation,
- Household water connections from the main supply,
- Compressed air lines.

11.10.7.2 Laying and Jointing

MDPE compression fittings made of PP, electrofusion fittings (PE 100), ABS, UPVC are available in India for use with MDPE pipes. The material used for the fittings shall also be suitable for conveying potable water. The joining material of compression fitting consist of virgin thermoplastic resin of polyethylene-type, NBR ‘O’ ring of nitrile and clamp of Polypropylene, copolymer body, zinc plated steel reinforcement ring, nuts, and balls of special NBR gasket. Push fit compression fittings are typically used for sizes 20–63 mm. The joining material of electrofusion fittings consists of virgin PE 100 material resin for permanent fusion jointing with MDPE pipe.

11.10.7.3 Advantages and Disadvantages

The advantages of pipes are:

- Very smooth inner surface; ensures no scaling and choking.
- Has less friction loss and gives better flow at lower heads.
- Easy to transport and store as the pipes available in 100, 200, and 300 m coils.
- No wastage of pipe as it can be cut to requirement at site.
- Less number of joints as the pipe is flexible and easy to repair.
- Easy tapping with specialty tapping joints.

- Tools-off installation possible with precision made fittings.
- Resistance to inorganic acids, alkalis and salts, hydrocarbon gasses.

The disadvantages of pipe are:

- Susceptible to chemical attack especially solvents.
- Strength may be affected by sunlight.

11.11 Glass Fibre Reinforced Plastic (GRP) Pipes

11.11.1 General

Glass fibre reinforced plastic (GRP) pipes are now being manufactured in India conforming to IS 12709: 1994 (Reaffirmed Year: 2019) – Glass fibre Reinforced Plastic (grp) Pipes Joints and Fittings for Use for Potable Water Supply. Five pressure classes of pipes namely, PN 3, PN 6, PN 9, PN 12, and PN 15 correspond to the working pressure ratings of 3.06, 6.12, 9.18, 12.24, and 15.30 kg/sq. m, respectively. Pipes sizes covered under this IS code are 200 mm to 3000 mm dia. Provisions relating to fittings fabricated from GRP pipes or by moulding process have also been covered.

Stiffness is the prime design criteria in the case of underground pipes. GRP pipe stiffness is classified into four classes, depending on the type of installation, overburden above the crown of the pipe and the soil conditions, GRP pipe stiffness is classified into four classes viz. A, B, C, D. The specials are made out of the same pipe material, i.e., glass fibre reinforced plastic (GRP).

GRP pipes are widely used in other countries where corrosion-resistant pipes are required at reasonable costs. GRP can be used as a lining material for conventional pipes, which are subject to corrosion. These pipes can resist external and internal corrosion whether the corrosion mechanism is galvanic or chemical in nature.

11.11.2 Laying

Pipes shall be supplied in nominal length of 6 m, 9 m, and 12 m. A maximum of 10 per cent of the pipe section may be supplied in random lengths. Lengths other than those specified may be supplied as agreed between the purchaser and the manufacturer. The tolerance on nominal lengths shall he within ±25 mm. Wall thickness shall be measured to an accuracy of 0.1 mm.

The width of the trench at top of the pipe should not be greater than necessary to provide adequate room for joining the pipe in the trench and for compacting the backfill in the zone of the pipe at the side thereof. If necessary, bell holes are permissible at the joints.

GRP pipes being light in weight, can be easily loaded or unloaded by slings, pliable stripes, or ropes. A pipe can be lifted with only one support point or two support points, placed about 4 metres apart. Excavation of trench and back filling of materials is similar to that in the case of CI and MS pipes.

The surface at the trench grade should be continuous, smooth, and free of big rocks more than 1.5 times the thickness of the pipe if rounded, or more than 1.0 times the thickness of the pipe if they have sharp edges and may cause point loading on the pipe. When ledge rock, hard pan, big rocks, timber, or other foreign materials are to be found, it is advisable to pad the trench bottom with sand or compacted fine-grained soils at least 150 mm thick so as to provide an adequate foundation.

The pipe should be uniformly and continuously supported through its whole length with firm stable bedding material. Pipe bedding material should be sand or gravel as per the requirements on the backfill material. The bedding should be placed so as to give complete contact between the bottom of the trench and the pipe and backfilling should be compacted to provide a minimum compaction corresponding to 90% maximum dry density. Lift should normally not be greater than 30 cm in height and the height differential on each side of the pipe should be limited to this amount so as to prevent lateral movement of the pipe.

Jointing

All joints installed or constructed in the field shall be assembled only by trained technicians. After the completion of pipe installation at site, the pipeline should be tested for 1.5 times the working pressure for 30 minutes with water. All pipe joints shall be watertight. All joints that are found to leak by observation or during testing shall be repaired and retested.

The pipes are joined as per the techniques: double bell coupling (GRP) for GRP to GRP, flange joint (GRP) for GRP to valves, CA pipes or flanged pipes, mechanical coupling (steel) for GRP to GRP/steel pipe and butt-strap joint (GRP) for GRP to GRP.

Pipes are joined by using double bell couplings in following manner.

(i) Double bell coupling grooves and rubber gasket rings should be thoroughly cleaned to ensure that no dirt or oil is present.
(ii) Lubricate the rubber gasket with the vegetable oil-based soap which is supplied along with the pipes and insert it in the grooves.
(iii) With uniform pressure, push each loop of the rubber gasket into the gasket groove. Apply a thin film of lubricant over the gaskets.
(iv) Apply a thin film of lubricant to the pipe from the end of the pipe to the back-positioning stripe.
(v) Lift manually or mechanically the double bell coupling and align with the pipe section.
(vi) Push the coupling onto the pipe by using levers. For large diameter pipe, the coupling may be pushed mechanically with even force on the coupling ring.
(vii) Apply a thin film of lubricant over the pipe to be pushed into the coupling just assembled until the stripes on the pipe are aligned between the edge of the coupling.

Thus, pipes are coupled together, and the rubber gasket acts as a seal making the joint leak proof. Joint types are normally adhesive bonded, however, reinforced overlay and mechanical types such as flanged, threaded, compressed couplings or commercial/proprietary joints are available.

Unrestrained joints of pipe capable of withstanding internal pressure but not longitudinal forces.

- Coupling or socket and spigot gasket joints provided with groove(s) either on the spigot or in the socket to retain an elastomeric gasket(s) that shall be the sole element of the joint to provide water tightness.
- Mechanical couplings.

Restrained joints capable of withstanding internal pressure and longitudinal forces,

- Joints similar to coupling or socket and spigot gasket Joints with supplemental restraining elements;
- Butt Joint – with laminated overlay;
- Socket and spigot – with laminated overlay;
- Socket and spigot – adhesive bonded;
- Flanged; and
- Mechanical.

11.11.3 Testing of Pipeline

Working hydraulic pressure in the system shall not exceed the pressure class of the pipe. When surge pressure is considered, the maximum pressure in the system, due to working pressure plus surge pressure, shall not exceed 1.4 times the pressure class of pipe. Other type of tests may be referred to IS 12709: 1994, Reaffirmed 2019.

11.11.4 Advantages and Disadvantages

The advantages of pipe are:

- High strength to weight ratio;
- Corrosion-resistant;
- Light weight compared to metallic and concrete pipes; and
- Longer length and hence minimum joints enable faster installation and very less chance of leaks.

The disadvantages of pipe are:

- High material cost;
- Brittle, require careful installation;
- High installation cost;
- Not suitable in rocky stretches and above ground installations.

11.12 House Service Connections

Distributing water with 100% consumer metering is most essential. Hence, consumer metering is necessary. Water supply to a house begins with connection of the service pipe with water supply mains. The Service connection pipe and internal plumbing shall conform to National Building Code or related IS code. The connections shall be made only by licensed plumber and be controlled/ vetted by NRW cell of ULB. Restoration/Reinstatement of the road crossings to its original state after making service connection should be included in the BOQ.

11.12.1 Laying and Jointing

11.12.1.1 Medium density Polyethylene Pipes (MDPE)

The medium density polyethylene pipes (MDPE) are now being manufactured in India conforming to ISO specifications (ISO 4427 and BS 6730 - 1986) for carrying potable water. However, no BIS is available for these pipes. The MDPE pipes are being used for consumer connection pipes as an alternative to GI pipes. The polyethylene material used for making the MDPE pipes conforms to PE 80 grade and the MDPE pipes when used for conveying potable water does not constitute toxic hazard and does not support any microbial growth. Further, it does not impart any taste, odour, or colour to the water.

The MDPE pipes are colour-coded black/blue with blue strips in sizes ranging from 20 mm to 110 mm diameter for pressure class of PN3.2, PN4, PN6, PN10, and PN16. The maximum admissible working pressures are worked out for temperature of 20^0C as per ISO 4427. The pipes are supplied in coils and minimum coil diameter is about 18 times diameter of the pipe.

MDPE compression fittings made of PE, PP, AABS, UP VC are also available in India for use with MDPE pipes. The materials used for the fittings are also suitable for conveying potable water like MDPE pipes. The jointing materials of fittings consists of thermoplastic resins of Polyethylene-type, NBR 'O' ring of Nitrile and Clamp of Polypropylene, copolymer body, Zinc plated steel reinforcing ring, nuts and balls of special NBR gasket.

The MDPE pipes are lightweight, robust, and non-corrodible and hence, can be used as alternative material for consumer connections. Since the pipes are supplied in coils, there will be no joints under the roads and bends are avoided resulting in fast, simple, and efficient jointing.

11.12.1.2 Polyethylene-Aluminium-Polyethylene (PE-AL-PE)

Polyethylene-Aluminium-Polyethylene (PE-AL-PE) conforming to IS-15450-2004 (revised in 2022) is suitable for HSCs. Multilayer composite pipe delivers new standards of quality in the field of water supply and is being used for HSC pipe as a better alternative. The PE-AL-PE multilayer composite pipe comprises one aluminium layer, tie layers of polymeric adhesive, and inner and outer layers of HDPE. The inner and outer polyethylene layers are bonded to metallic aluminium layer by polymeric adhesive during manufacturing of pipe. It is light, strong and does not support corrosion. BIS has approved PE-AL-PE. Pipes for dia (outer dia) ranging from 14 mm to 75 mm which may be used in service connection pipes for commercial and industrial establishments and bulk demand for residential complexes/group of households. Figure 11.37 shows PE-AL-PE pipe.

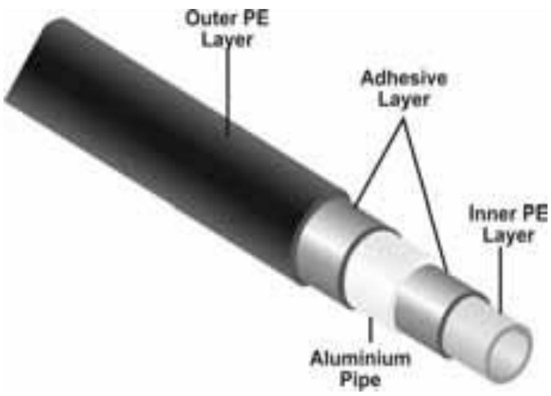


Figure 11.37: PE-AL-PE Pipe

These pipes have non-corroding thermoplastic layers that resist the most aggressive water conditions and hot soil environments. The PE-AL-PE composite pipe are pressure rated for maximum water pressures of 13.8 Kg/cm^2 at 23 °C, 11 Kg/cm^2 at 60 °C and 6 Kg/cm^2 at 80°C. With a Hazen-Williams flow coefficient of C-150, these pipes will not corrode or allow algae build-up inside the pipe which can increase friction losses. Long-term pressure rating of these pipe includes safety factor of 2:1. Hence, the PE-AL-PE pipes easily handles pressure increases created by surges in a water service application. Figure 11.38 shows Jointing of PE-AL-PE pipe.

The PE-AL-PE pipe has resistance to chlorine attack than other non-composite pipes because of the aluminium middle layer. These pipes are lightweight, robust, and non-corrodible and hence, can be used as alternative material for consumer connections.

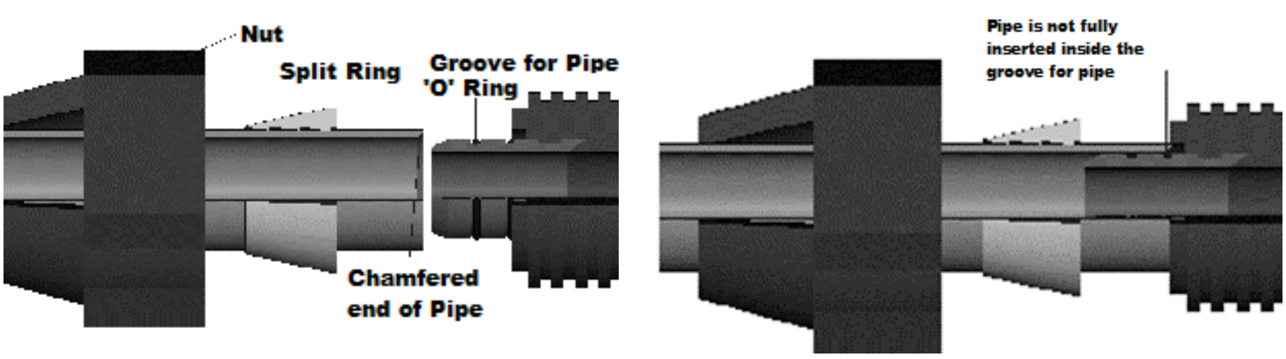


Figure 11.38: Jointing of PE-AL-PE Pipe

11.12.2 Saddle sets in HSCs

Major portion of leakages occur in HSCs. At present, saddle set with female threaded nipple is used in HSC and this practice results into high leakages at the ferrule points and contamination of water in the pipeline. These leakages from HSC should be controlled and reduced by using leak-proof saddle sets and good quality HSC pipes. There are two types of saddles used in the distribution system having plastic pipes (PVC/HDPE) for HSC, one is mechanical PP saddle set which is used for both PVC/HDPE pipelines and other is fusion welded Electrofusion saddle set for HDPE pipeline.

On PVC/HDPE pipelines, the mechanical PP clamp saddle (IS 7634 Part 2-2012, Reaffirmed 2022 Fig. 15A) with inbuilt compression fittings manufactured using injection moulded integrated saddle which can be sealed on these pipes by rubber gasket/sealing. The service saddle is manufactured out of alloy of virgin polypropylene (PP) and is made with special additives that provide UV protection. It has higher mechanical strength and ensures smooth flow. The service saddle is available in black/blue colour. The PP service saddle set is used for PVC and HDPE pipes. These saddles are suitable for HSCs in urban areas. Figure 11.39 shows monolithic service saddle for non-metallic pipes.

On HDPE pipeline, fusion welded electrofusion tapping saddle is installed as per IS 7634-2-2012, Reaffirmed 2022 with inbuilt cutter for tapping HDPE pipe. The electrofusion saddle set shall be made of polyethylene (PE 100) and is made with special additives that provide UV protection. Electrofusion Saddle set as a new technology, fused on HDPE pipe by fusion process which installed without stopping main line and it is leak proof as the joints are fusion welded. The saddle set has both top and bottom part which are screwed on HDPE pipe. The fusion joint made with integrated electrofusion tapping saddle makes complete piping system homogenous and there is no possibility of any contamination of water due to high leak-proof nature. Electrofusion tapping saddle integrated with 8mm cutter shall be used in the pipes in which residual pressure is more than 7 m (0.7 kg/cm2) and 15 mm cutter in the pipeline having residual pressure less than 7 m (0.7 kg/cm2). These saddles are also suitable for HSCs in urban areas. The electrofusion tapping saddle is shown Figure 11.40. Typical HSC on HDPE pipe with Electrofusion saddle is given in Figure 11.41.

On a metal pipeline, service saddle made of composite polypropylene glass reinforced fibre or DI material is used. The strap saddle is used up to 200 mm dia metallic pipes. and Figure 11.42 shows strap saddle for metallic pipes.

It must be ensured that all service saddles should be fitted with bottom part for effective clamping and leak proof.

Figure 11.39: Monolithic Service Saddle for non-metallic pipes

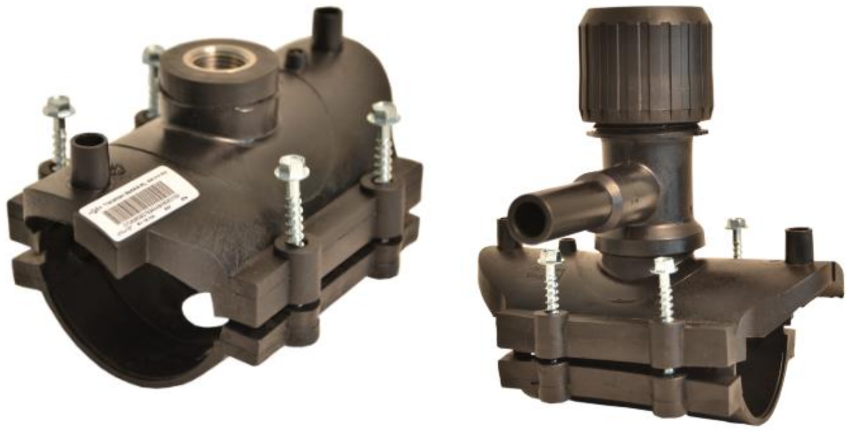


Figure 11.40: Electrofusion Tapping saddle with integrated cutter, Electrofusion transition saddle for HDPE pipe

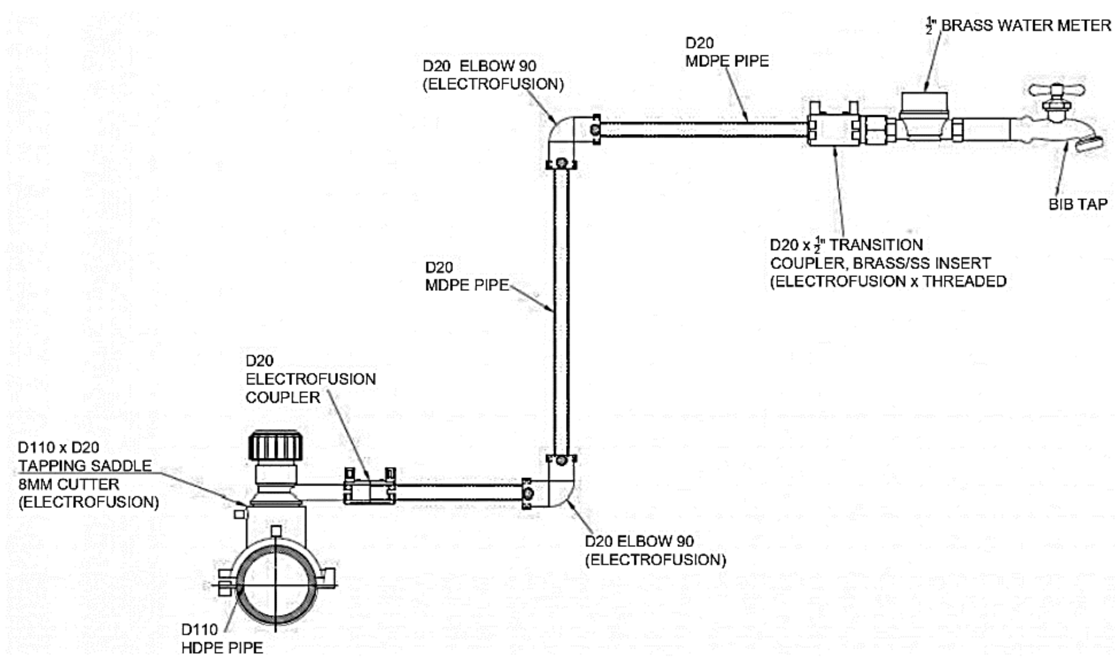


Figure 11.41: The complete HSC is given in figure

Figure 11.42: Strap Saddle for metallic pipes

Department of Drinking Water Supply, Ministry of Jal Shakti has recommended adoption of integrated clamp saddle set with flow control valve (FCV) in HSC in rural areas under Jal Jeevan Mission 2 to ensure equity and efficiency in water supply services. The integrated saddle with FCV is designed for 5 LPM (± 0.75 lpm) discharge at 0.50 pressure and not exceeding the flow of 7 LPM (± 0.75 LPM) at 1 bar (10 m head) pressure. The integrated saddle set is manufactured using injection with FCV SS316 solid steel bar. FCV/NRV can be designed for higher flow control discharge beyond 7 LPM subject to distribution design for effective water needs beyond 7 LPM. The terminal pressure of 7 m is recommended in rural water supply distribution network where no FCV is used in the HSCs and 12 m is recommended in the rural water supply distribution network, where FCV is used in the HSC, considering the head loss in the FCV at the ferrule point.

However, the integrated saddle set with FCV may be avoided in the HSCs in urban areas as it may lead to increased head loss at the ferrule point and affect the recommended terminal pressure of 17–21 m for Class I and II cities and 12–15 m for Class III–VI towns.

11.13 Aspects of Plumbing System

Bureau of Indian Standards has issued a handbook on water supply and drainage with special emphasis on plumbing vide IS: SP-35**:** 1987 (Reaffirmed Year: 2014), which shall generally be followed along with relevant latest BIS codes of practices.

Pipe Materials in Plumbing:

The BIS standards IS: 2065**:** 1983 (Reaffirmed Year: 2022) – Code of practice for water supply in buildings covers general requirements and regulations for water supply, plumbing connected to public water supply, licensing of plumbers, design of water supply systems, principles of conveyance and distribution of water within the premises, storage, water fittings and appliances, and inspection and maintenance. Necessary in-line valves at appropriate places have to be installed for control and isolated repairing services in a particular stretch of plumbing piping system. This code does not cover aspects of water supply for firefighting purposes, which are covered under different IS codes.

The BIS specification code IS 12183-1: 1987 (Reaffirmed Year: 2019) Code of Practice for Plumbing in Multi-storeyed Buildings: Part 1 Water Supply deals with water supply in multi-storeyed buildings and covers general requirements and regulations, design considerations, plumbing systems, distribution systems, storage of water, and inspection for water supply in multi-storeyed buildings. Wherever, vertical separation of floors in multi-storeyed buildings is possible, necessary control valve(s) at each floor shall be provided for equitable pressure and flow. Also, in buildings where vertical separation is not possible, pressure in the lower floors may be restricted by use of pressure reducing valves, orifice flanges, or other similar devices. The BIS code recommended that the velocity of water in pipes should be restricted to 2.0 m/s to avoid noise problem. It is recommended to consider a minimum residual head of 21 m at the ferrule at the highest spot of operational zone for Class I and II cities and 15 m for other towns. Care should be taken to obtain the flow required for the minimum pressure at all parts in the building.

Requirements for water piping, fittings and appliances, inspection, and maintenance covered in IS: 2065 (1983, (Reaffirmed Year: 2022)) shall also be applicable for buildings. All pipe runs, appurtenances, and valves shall be located in a manner to provide easy access for maintenance and repair.

As per the IS code, plumbing refers to (a) the pipes, fixtures, and other apparatus inside a building in the water supply and removing the liquid and waterborne wastes; (b) the installation of foregoing pipes, fixtures, and other apparatus. The plumbing system refers to and includes the water supply and distribution pipes, plumbing fittings and traps, soil, waste, vent pipes, and anti-siphonage pipes, building drains and building sewers, including their respective connections, devices, and appurtenances within the property lines of the premises, and water-treating or water using the equipment.

Premises includes passages, buildings, and lands of any tenure, whether open or enclosed, whether built on or not, and whether public or private, in respect of which a water rate or charge is payable to the authority or for which an application is made for the supply of water.

Commonly terms used to construct water pipes in plumbing are:

a) Supply pipe – Pipe connecting to municipal water mains to HSC;
b) Vertical pipe – Any pipe which is installed in a vertical position or which makes an angle of not more than 45° with the vertical;
c) Warning pipe – An overflow pipe so fixed that its outlet, whether inside or outside a building, is in a conspicuous position where the discharge of any water therefrom can be readily seen;
d) Stopcock – A cock fitting in a pipeline for controlling the flow of water;
e) Stop tap – Stop tap includes a stopcock, stop valve, or any other devices for stopping the flow of water in a line or system of pipe at will;
f) Storage cistern – A cistern for storing water;
g) Water line – A line marked inside a cistern to indicate the highest water level at which the supply valve should be adjusted shutoff;
h) Water main (street main) – A pipe laid by the water undertakers for the purpose of giving a general supply of water as distinct from a supply to individual consumers and includes any apparatus used in connection with such a pipe;
i) Water outlet – A water outlet, as used in connection with the water distribution system, is the discharge opening for the water (a) to a fitting, (b) to atmospheric pressure (except into an open tank which is part of the water supply), and (c) to any water-operated device or equipment requiring water to operate;
j) Washout valve – A device located at the bottom of the tank for the purpose of draining a tank for cleaning, maintenance, etc.
k) Water supply system – Water Supply System of a building or premises consists of the water service pipe, the water distribution pipes, and necessary connecting pipes, fittings, control valves, and all appurtenances in or adjacent to the building or premises.

Figure 11.43 shows Sketch showing typical Bedding Angle.

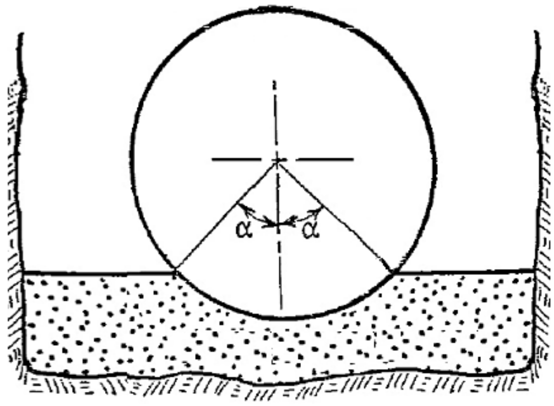


Figure 11.43: Sketch showing typical Bedding Angle

Bedding and bedding angle – The bedding is the material placed in the bottom of the trench on which the pipe is laid. The bedding for both rigid and flexible pipe is an uncompacted layer of select material. This layer of uncompacted select material is placed over the foundation or the replaced foundation. The thickness of this layer depends on the pipe diameter. For pipe with a diameter of 300 to 1350 mm (12 to 54 in), the thickness of the bedding is 100 mm (4 in). For pipe diameters larger than 1350 nun (54 in), the thickness of the bedding is 150 mm (6 in). Pipe is laid directly on the bedding. The pipe bedding factor is very important as this adds a great deal of strength to the pipe when it is in service. Ideal factor can be anywhere between 90% and 120% or more of the total structural strength of the pipe/bedding system. Pipe Bedding Class A (Plain Concrete Cradle) includes an unreinforced concrete cradle encompassing 120° of the bottom of the pipe. Fine grading of the surface of the bedding shall be such that the final grade of the pipe shall not exceed the specified departure from grade. Because when the bedding material is uncompacted, there will be some slight settlement of the pipe laid on the uncompacted bedding. The amount of settlement will vary depending on the type of soil, the type of pipe, and the diameter of the pipe. The initial (placement) thickness of the bedding layer will have to be established by trial and error at the beginning of a job or after any change that would affect the settlement. If the bedding becomes compacted by excessive foot traffic, equipment travel, or rain, before the pipe has been placed, it must be loosened by removal and replacement or by scarifying. For special designs of very large diameter pipe, a thicker bedding may be specified.

Proper design of the water distributing systems in a building is necessary in order that the various fittings may function properly, and there is an adequate supply to meet the needs of the occupants of the building, both with regard to their domestic as well as flushing (of sanitary appliances) requirements. Pipes going underground must have adequate cover. As per the relevant IS code, domestic water metres should be installed as prescribed by the authority to receive potable water. Figure 11.43 shows sketch of typical bedding angle.

11.13.1 Polypropylene-Random Copolymer Pipes for Hot and Cold Water

The BIS Code IS 15801 (2008, (Reaffirmed Year: 2022)) – Polypropylene-random copolymer pipes for hot and cold water supplies specifies requirements for polypropylene-random copolymer pipes from 16 mm to 200 mm nominal diameter of standard dimension ratio (SDR) 11, 7.4, 6, and 5 for:

a) wall concealed hot and cold water conveyance pipelines for inside and outside buildings (properly UV stabilised), and
b) pipelines for the solar heating system inside and outside the buildings.

Standard dimension ratio (SDR) is a method of rating a pipe's durability against pressure. The standard dimension ratio describes the correlation between the pipe dimension and the thickness of the pipe wall. Common nominations are SDR11, SDR7.4, SDR6, SDR 5. Pipes with a lower SDR can withstand higher pressures.

Polypropylene-random copolymer (PP-R) used for the manufacture of pipes shall conform to the requirement of IS 10951 (2002): Polypropylene Materials for Moulding and Extrusion and IS 10910 1994 (Reaffirmed 2003) IS 10910 (1984): Polypropylene and its copolymers for its safe use in contact with foodstuffs, pharmaceuticals and drinking water. The PP-R pipes, when in permanent or temporary contact with water, which is intended for human consumption, shall not adversely affect the quality of drinking water.

11.14 Pipeline in Colder Region

Selection of pipe material in colder climate shall take into consideration the BIS Code IS 6295: 1986 (Reaffirmed Year: 2022) Code of practice for water supply and drainage in high altitudes and/or sub-zero temperature regions to handle the following issues:

- Freeze-Back Forces: Excessive pore water pressure trapped between the permafrost layer and winter frost layer can deform or collapse pipe.
- Potential Freeze Damage: Pipes should, if freezing risk is high, be capable of being thawed and returned to service without loss of strength.

The general recommended type of pipe materials are HDPE, AC, GI, CI, and uPVC as per "IS 6295: 1986 (Reaffirmed Year: 2022) – Code of practice for water supply and drainage in high altitudes and/or sub-zero temperature regions". Ductile iron can also be used as per the relevant ISO Standards 21051:2020.

Unless piping can be installed below the seasonal frost line, some form of freeze protection/insulation is mandatory. Factory-applied polyurethane is recommended for both buried and over ground pipes.

UPVC pipe becomes brittle in the cold and can shatter when frozen. PVC is not recommended for use where there is a chance of the pipe being exposed to freezing conditions. UPVC pipe shall be buried to depths unlikely to experience freezing temperatures.

HDPE is preferred where there is a high risk of freezing. HDPE can normally be thawed and returned to service without damage to the pipe. It is corrosion-resistant and not affected by extreme cold.

The BIS Code "IS 6295 (1986, Reaffirmed 2022): Code of practice for water supply and drainage in high altitudes and/or sub-zero temperature regions" gives recommendations regarding the factors to be given consideration while planning and designing water supply and sanitation system peculiar to high altitudes and/or sub-zero temperature regions of the country.

In transmission and distribution, freezing of the buried pipe may be avoided primarily by laying the pipe below the level of the frost line; well consolidated bedding of clean earth or sand, under, around or over the pipe should be provided. The level of frost line is generally found to be between 0.9 m and 1.2 m below ground level in the northern regions of India, wherever freezing occurs.

For the efficient operation and design of transmission and distribution work, the available heat in the water shall be economically used and controlled. If heat naturally present in water is inadequate to satisfy heat losses from the system, the water shall be warmed. Where economically feasible, warm water from hot springs or other ground water sources, if potable, shall be mixed with the primary source for this purpose. If found unsuitable for drinking water purposes, such water may be used for heating purposes. Heat losses shall be reduced by insulation, if necessary. Any material that will catch, absorb, or hold moisture shall not be used for insulation purposes. Adequate number of break pressure water tanks and air release valves shall be provided in the distribution system. HDPE pipes with proper break pressure chambers along with the outlets at suitable positions will prove to be a good transmission/distribution system. Where high density polyethylene (HDPE) pipes are used in some cases cylinder of ice is formed inside the pipe near the joints. Extra precaution should be taken near the joints by way of lagging.

Materials for insulation of pipes as per the normal practice in India is to surround the pipe with straw, grass, hessian cloth/strip or jute rapped over with gunny and painting with bitumen; alternatively, other materials like 85 per cent magnesia, glass wool or asbestos coated lagging ropes may also be used.

HSCs shall be kept operative by the use of adequate insulation at exposed places extending below the frost line. A typical arrangement of providing insulation for HSCs is shown in Figure 1 of the BIS code IS 6295 (1986, Reaffirmed 2022), which may be referred to for general guide, wherein guideline for fire hydrant protection in freezing climate region is also provided.

11.15 Excavation and Preparation of Trench

Excavation may be done by hand or by machine. The trench shall be so dug that the pipe may be laid to the required gradient and at the required depth. When the pipeline is under a roadway, a minimum cover of 0.9 m is recommended for adoption, however cover should be provided as per respective BIS code for different pipe materials & suiting to the local field conditions by taking necessary precautions. However, the structural strength of the pipe, based on dead load and live load over the pipe, should also be analysed. The trench shall be so braced and drained that the workmen may work therein safely and efficiently. The discharge of the trench dewatering pumps shall be conveyed either to drainage channels or to natural drains and shall not be allowed to be spread in the vicinity of the worksite. The width of the trench at the bottom shall provide not less than 200 mm clearance on both sides of the pipe. Additional width shall be provided at positions of sockets and flanges for jointing. Depths of pits at such places shall also be sufficient to permit the finishing of joints.

Ledge rock, boulders, and large stones shall be removed to provide a clearance of at least 150 mm below and on each side of pipes in case of valves and fillings for pipes of 600 mm diameter or less and 200 mm for pipes larger than 600 mm in diameter.

11.16 Shoring and Strutting

The shoring shall be adequate to prevent caving in of the trench walls by subsidence of soil adjacent to the trench. In narrow trenches of limited depth, a simple form of shoring shall consist of a pair of 40 to 50 mm thick and 30 cm wide planks set vertically at intervals and firmly fixed with struts. For wider and deeper trenches, a system of wall plates (Wales) and struts of heavy timber section is commonly used. Continuous sheeting shall be provided outside the wall plates to maintain the stability of the trench walls. The number and the size of the wall plates shall be fixed considering the depth of the trench and the type of soil. The cross struts shall be fixed in a manner to maintain pressure against the wall plates, which in turn shall be kept pressed against the timber sheeting by means of timber wedges or dog spikes. In non-cohesive soils combined with considerable groundwater, it may be necessary to use continuous interlocking steel sheet piling to prevent excessive soil movements by groundwater percolation and extend the piling at least 1.5 m below the trench bed. In the case of deep trenches, excavation and shoring may be done in stages.

11.17 Handling of Pipes

While unloading, pipes shall not be thrown down but may be carefully unloaded on inclined timber skids. Pipes shall not be dragged over other pipes and along concrete and similar pavements to avoid damage to pipes.

11.18 Detection of Cracks in Pipes

The pipes and fittings shall be inspected for defects and be rung with a light hammer, preferably while suspended, to detect cracks.

11.19 Lowering of Pipes and Fittings

All pipes, fittings, valves, and hydrants shall be carefully lowered into the trench by means of derrick, ropes, or other suitable tools and equipment to prevent damage to pipe materials and protective coatings and linings. Pipes over 300 mm in diameter shall be handled and lowered into trenches with the help of chain pulley blocks.

11.20 Anchorages

A pipeline need anchorage at dead ends and bends as appreciable thrust occurs, which tends to cause draw and even "blow out" joints. Where thrust is appreciable, concrete blocks/restrained joint system should be installed at all points where movement may occur. Anchorages are necessary to resist the tendency of the pipes to pull apart at bends or other points of unbalanced pressure or when they are laid on steep gradients and the resistance of their joints to longitudinal or shear stresses is either exceeded or inadequate. They are also used to restrain or direct the expansion and contraction of rigidly joined pipes under the influence of temperature changes. Anchor blocks shall be designed in accordance with IS: 5330 (1984, Reaffirmed 2020).

11.21 Thrust Blocks

Water travelling through a piping system under internal pressure exerts a thrust force at all bends, tee junctions, and stop ends. The magnitude of these forces usually is so high that they can easily weaken the joints and even can cause leakage or failure of the piping/pipeline system. With an increase in the piping size, these forces increase further. Installation of a thrust block partially absorbs that pressure thrust force and the remaining is transferred to the surrounding soil. As such, thrust blocks to be designed and cast at all the horizontal bends, vertical bends, tees, wye's, reducers, and enlargers.

The following are some typical thrust force calculation formulas for ductile iron pipes.

- **Thrust Force on an Elbow or bend**

  To calculate the design thrust force or resultant force for bends the following formula can be used:
  Thrust force, $F = 2 P A \sin(\phi/2)$
  Where: P = design pressure,
  A = cross-sectional area of the pipe, and
  ϕ = angle of the bend.
- **Thrust force on Plugs or Caps**
  The thrust force in a plug or cap is equal to the design pressure (P) times the cross-sectional area (A) of the pipe.
  (Thrust force, $F = P A$).
- **Thrust force for tee connections**
  The thrust force generated in a Tee connection is calculated as
  $F = P A_b$.
  Where P=internal design pressure and
  A_b= cross-sectional area of the branch pipe.
- **Thrust force calculation of pipe reducers**
  The design thrust force for piping reducers/expanders is equal to the design pressure (P) times the difference of the cross-sectional areas of the large (A_1) and small end (A_2) sizes of the reducer.
  Hence, thrust restraint force, $F = P (A_1 - A_2)$

11.22 Bore well / Tube well

A Borewell/Tube well construction and assembly consist of the following pipes mainly:

a) **Casing Pipe:** A casing pipe is a pipe that is used to protect the wells and the boreholes from collapsing.
b) **Housing Pipe:** The housing pipe is the upper portion of the case section of the well and serves as housing for the pumping equipment and is a vertical conduit through which water flows from the aquifer to the pump. It is watertight and extends downwards from the ground surface to a safe depth below the anticipated pumping water level.
c) **Drive pipe:** Drive pipe is also a type of casing made of seamless or welded mild steel pipes designed to withstand the driving force and penetrate into the ground to protect against the collapse of the movement of the loose formation which takes place during drilling operations.

11.22.1 Casing/Housing/Drive Pipes

Mainly following two types of pipes are used for tube well casing/housing/drive:

I) Steel: General requirements relating to the supply of steel tubes for water wells shall conform to IS 4270 (2001, Reaffirmed 2022): Steel Tubes Used for Water Wells [MTD 19: Steel Tubes, Pipes and Fittings]

II) uPVC shall conform to IS 12818 (2010, Reaffirmed 2021) Unplasticized Polyvinyl Chloride (PVC-U) screen and Casing Pipes for Bore/tube well

I) **Steel tubes**: Steel tubes can be classified based on the manner in which it is manufactured as follows:

a. Automatic fusion welded pipe: A tube made from steel plates formed into the pipe and welded longitudinally by a submerged arc welding process.
b. Electric resistance welded pipe: Electrically welded tube is made from steel strip which is formed into a tubular shape and welded by passing a heavy current across the longitudinal joint.
c. High-frequency induction welded pipe: The electrically welded tube is made from a steel strip which is formed into a tubular shape and welded by passing a current at a high frequency across the longitudinal joint.
d. Seamless steel tube or pipe: A tube without a longitudinal joint or weld.

Steel tubes shall be one of the following types and grades of steel:

1. Hot finished seamless (HFS),
2. Electric fusion welded (EFW),
3. Electric resistance welded (ERW), and
4. High-frequency induction welded (HFIW).

Grade of steel: Fe 410 and Fe 450 are generally used to manufacture steel pipes for borewell/tube well.

II) **uPVC Pipes:** Follow the latest IS: 12818 (2010, Reaffirmed 2021) Unplasticized Polyvinyl Chloride (PVC-U) screen and Casing Pipes for Borewell/tube well, which cover DN 35 mm to DN 400 mm for borewells/tube wells for water supply. Solid wall plain surface pipes are used as extension pipes to the screen pipe.

Casing pipes are classified into three categories based on well depth that are shallow, medium, and deep described below:

i. CS (Casing for shallow well) pipe: Shallow well casing pipes suitable for wells with depths up to 80 m.
ii. CM (Casing for medium well) pipe: Medium well casing pipes suitable for wells with depths beyond 80 m and up to 250 m.
iii. CD (Casing for deep well) pipe: Deep well casing pipe suitable for wells with depths beyond 250 m and up to 450 m.

11.22.2 Screens and Slotted Pipes:

Borewell/tube well screens and slotted pipes shall conform to IS 8110: 2019 – Water Well Screens and Slotted Pipes — Specification. In this Indian standard, FRP pipes have been covered for the manufacture of screens and slot pipes. Well screen is the most critical element in a tube well, affecting its life, pump maintenance, and efficiency.

Well screens are specially fabricated screen pipes from different materials which can have a wider range of slot opening from much finer to coarse compared to slotted pipes.

Slotted pipes are pipes with slots cut into them in a pattern suitable to the basic material of the pipe.

Following are the types of well screens and slotted pipes:
a) Plain slotted pipes: These are pipes with slots cut by milling;
b) Bridge slotted pipes: The slots here in the pipe are not cut but pressed out;
c) Mesh wrapped screens: These are made by wrapping copper mesh over perforated steel pipe using spacers about 3 mm thick in between the copper mesh and perforated pipe;
d) Cage-type wire wound screen: These are a special type of screen where a continuous shaped profile wire is spirally wound around a series of longitudinal support rods of a circular shaped section with each interaction of profile wire and support rod welded by electric fusion welding process. The longitudinal support rods are welded to end rings at both ends to facilitate joining with casing pipes or other screens by butt welding or threading;
e) Pre-packed resin bonded gravel screens: Gravel is pasted on the perforated pipe with the help of resin-type adhesive material. The thickness of the gravel bond varied between 10 mm to 15 mm depending upon the diameter of the base pipe;
f) Brass Screens: Brass screens are made from the brass sheet in which slots of required sizes are cut before rolling;
g) Ribbed Screen pipes: Pipe with external longitudinal ribs and transverse (perpendicular to pipe longitudinal axis) slots. This shall be designated as ribbed medium well screen (RMS) and ribbed deep well screen (RDS) pipes;
h) Plain screen pipes: These are plain surface pipes with transverse slots. This shall be designated as plain medium well screen (PMS) and plain deep well screen (PDS) pipe;
i) FRP strainers and slotted pipes: These are also being manufactured in India and are covered under IS: 8110: 2019. The FRP slotted pipes shall be manufactured by the filament winding process. The slots shall be cut by milling. The slotting patterns shall be such as to ensure the minimum cutting of the glass fibre in the pipe and thereby maintaining maximum strength. Selecting FRP slotted pipes made from FRP base material shall fulfil the requirements given in Annexure A of IS: 8110: 2019, which must be capable to withstand the internal hydrostatic pressure, external collapse pressure test, and water absorption and retention of strength.

Materials for well screens and slotted pipes shall be made of either corrosion-resistant material or steel pipes having sufficient thickness to guard against the effect of corrosion and to ensure reasonable life of the tube well.

The following are the recommended materials for various types of well screens and slotted pipes:

a) Low carbon or mild steel corresponding to IS: 1239 Part 1 (2004)
b) Leaded brass sheet corresponding to IS: 531 (1981, Reaffirmed 2006)
c) Fibre glass reinforced thermosetting plastics corresponding to IS: 12709 (1994, Reaffirmed 2009)
d) UPVC conforming to IS: 10151 (1982, Reaffirmed 1997)
e) Copper wire conforming to IS: 4412 (1981, Reaffirmed 2006)
f) Galvanised steel wire as per IS: 280 (2006)
g) Stainless steel wire as per IS: 6528 (1995, Reaffirmed 2001)

11.22.3 Joints

Casing/Housing/Drive Pipes joints shall be as follows:

a) Screwed and socketed butt joints;
b) Screwed flush butt joints; and
c) Plain bevelled end pipes for butt welded joints.

Well screens shall be threaded and socketed, plain bevel ended, collared or male and female types so that convenient lengths could be added. The slotted pipe screens shall have adequate strength to withstand axial, collapse, and hydrostatic loads to be experienced during development and use. The screen shall be as far as possible of single metal construction to avoid galvanic corrosion.

11.23 Appurtenances

The following appurtenances are used in the piping system:

1) Valves
2) Manholes/Inspection and repair chamber
3) Fire hydrants
4) Water Metres

11.23.1 Valves

Valves are mechanical devices that are operated manually or automatically to facilitate in operation and maintenance of water system or processes by control of flow, pressure, and formation of zones. Valves are also used to isolate and drain pipe sections for test, installation, cleaning and repairs. A number of appurtenances or auxiliaries are generally installed in the line. Various types of valves are used which have specific function within a system or process are described as follows:

11.23.1.1 Line Valves

Main line valves are provided to stop and regulate the flow of water in the course of ordinary operations and in an emergency. There are many types of valves for use in pipeline, the choke of which depends on the duty. The spacing varies principally with the terrain traversed by the line. In urban areas with connections in the distribution system, main aim is to sectionalise the line in order to maintain reasonable service. Figure 11.44 shows typical pipeline layout showing location of gate valve, air valves, and blow-off (drain) valves in a pipeline.

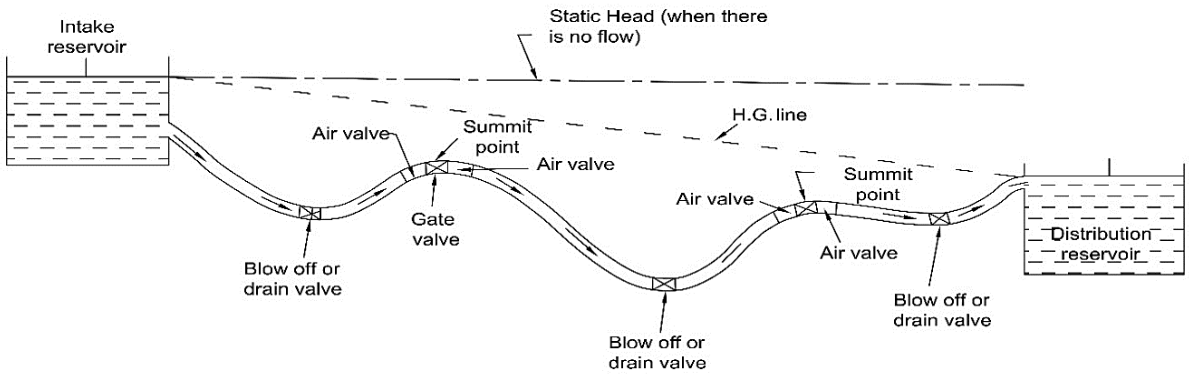


Figure 11.44: Typical Pipeline layout showing location of Gate Valve, Air Valves, blow-off (Drain) Valves in a pipeline

In larger lines isolating valves are frequently installed at intervals of 1 to 5 Km. The principal considerations in location of the valves are accessibility and proximity to special points such as branches, stream crossings, etc. The spacing of valves is a function of economics and operating problems. Sections of the pipeline may have to be isolated to repair leaks. The volume of water which would have to be drained to waste would be a function of spacing of isolating valves.

These valves are usually placed at major summits of pressure conduits. Summits identify the sections of the line that can be drained by gravity, and pressures are least at these points permitting cheaper valves and easier operation. Gravity conduits are provided with valves at points strategic for the operation of supply points, at the two ends of sag pipes and wherever it is convenient to drain the given section.

Normally, valves are sized slightly smaller than the pipe diameter and installed with a reducer on either side. In choosing the size; the cost of the valve should be weighed against the cost of head loss through it, although in certain circumstances it may be desirable to maintain the full pipe bore (to prevent erosion or blockage).

It is sometimes advisable to install small diameter bypass valves around large diameter in-line valves to equalise pressures across the gate and thus facilitate opening.

Various types of Line Valves are classified as below:

(i) Valve types based on functions

The valve serves various functions within the piping system. Such as

- Stopping and starting a fluid flow. Depending on whether a valve is open or closed, it let pass the process fluid or halt the fluid.
- Throttling the fluid flow. Some of the valves let you throttle the fluid depending open % of the total opening. Lesser the opening higher the throttling and otherwise.
- Controlling the direction of fluid flow. The multi-port valve lets you decide the way fluid will go.
- Regulating a flow or pressure within the piping system. Some of the automatic control valves maintain the flow and pressure within the system by adjusting opening and closing.
- Relieve pressure or vacuum from the piping system and equipment. Pressure and vacuum relief valve safeguard the process system from overpressure and during vacuum conditions.

Different types of valves serve these functions. These valves can be classified or categorised based on:

- Function
- End connection
- How it operates
- Types of actuators it used

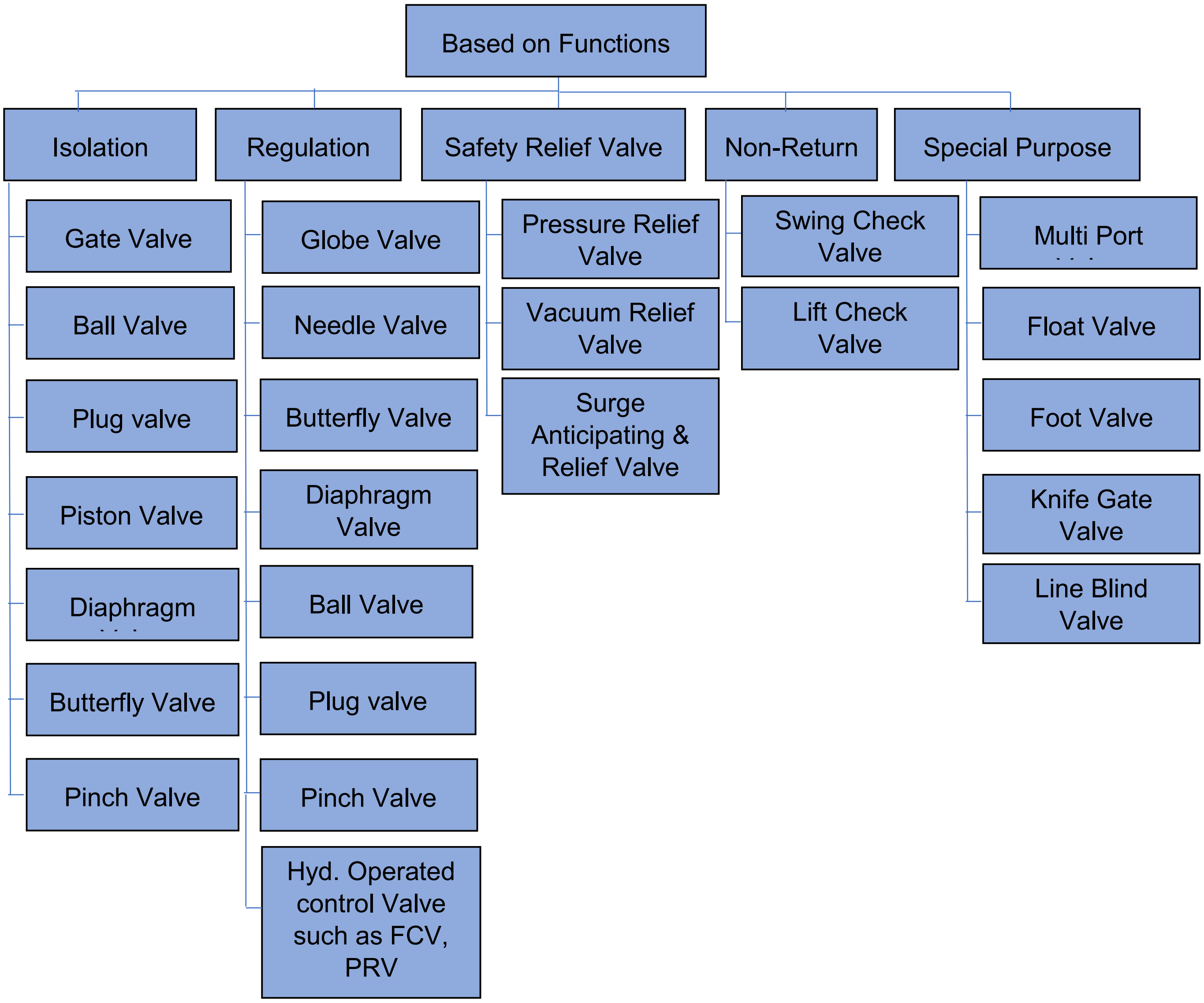


Figure 11.45: Classification of Valves Based on Function

In the above chart, you can see the types of valves and their function.

Isolation valve isolates or cuts the supply of fluid when needed. Gate, ball, plug, piston, diaphragm, butterfly, and pinch valve falls under this category.

A control valve that regulates the flow of fluid falls in the regulation category. Globe, needle, butterfly, diaphragm, ball, plug, and pinch valve are used as a control valve. You can see that some valves serve dual purposes such as the globe and the ball valve can be used as isolation as well as a control valve. Figure 11.45 shows classification of valves based on function.

Pressure and vacuum relief valve used to prevent overpressure and vacuum with the system that can damage the piping and equipment. Non-return valve such as swing and lift check valve prevents backflow within the system, whereas some valves are designed to serve a special purpose such as multi-port, knife, and line blind valve. Figure 11.46 shows classification of valves based on end connections.

Figure 11.46: Classification of Valves Based on End Connections

(ii) Valve type based on the end connection

Valve ends can be:

- Screwed or threaded that connect with matching thread on the pipe. A small bore valve used in instrument connection or as a sample point has a threaded end.
- The majority valve used in piping has a flanged-type end.
- Butt welded valves are used in very high pressure and temperature services.
- Socket welded valves are used in low pressure.
- Check valve and butterfly valves are available in wafer and lug end construction. These types of ends are used when space is constrained.

(iii) Valve type based on the way it is opened and closed

Another way to classify the valve is the way it opens and closes. Each valve opens and closes by either Linear or rotary motion or by the quarter turn which is nothing but a rotary motion.

a. Linear motion valves

Linear motion valves use a closure member that moves in a straight line and cut the flow to start, stop, or throttle the flow. The closure device could be a disc, or flexible material, such as a diaphragm. Linear motion valves are slower in operation, but they provide a higher level of accuracy and stability in the position of the closure member. Figure 11.47 shows linear motion valve.

b. Rotary motion valves

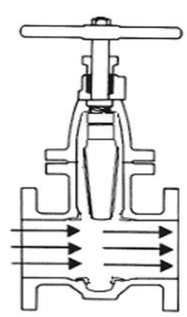


Figure 11.47: Linear Motion Valve

c. Rotary motion valves

These rotate a disc or swing it from the hinge pin that holds the disc. Figure 11.48 shows rotary motion valve.

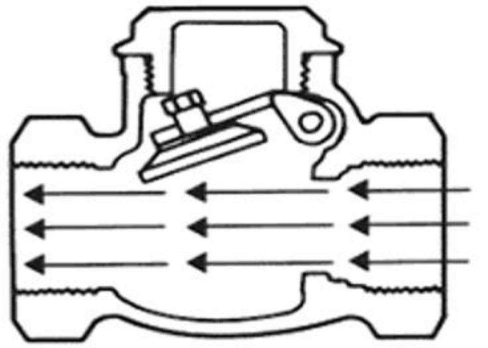


Figure 11.48: Rotary Motion Valve

d. Quarter turn valves

A 90° turn of the stem in quarter turn valves fully open or fully closed the valve. Because of this quick turn, the operation of the quarter turn valve is much faster than linear motion valves. Some rotary motion valves are also known as the quarter turn valve. Figure 11.49 shows quarter turn valve.

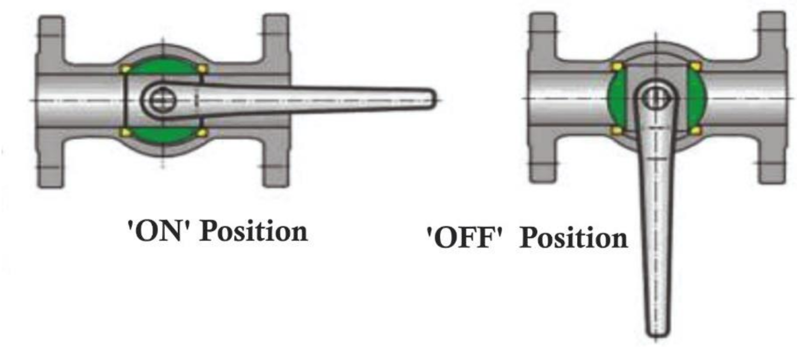


Figure 11.49: Quarter Turn Valve

Table 11.3: Motions of Different Types of Valves

Motion Type to open and close Valve	Valve Type
Linear Motion	Gate Valve
	Globe Valve
	Lift Check Valve
	In-Line Check Valve
	Stop Check Valve
	Pinch Valve
	Diaphragm Valve
	Safety Valve
	Relief Valve
	Swing Check Valve
	Tilting Disc Check Valve
	Folding-disc Check valve

Motion Type to open and close Valve	Valve Type
Rotary Motion	Stop Check Valve
	Ball Valve
	Butterfly Valve
	Plug Valve
Quarter Turn	Ball Valve
	Butterfly Valve
	Plug Valve

The table shows that the ball valve, butterfly valve, and plug valve are both rotary and quarter turn valves, whereas swing check, tilting disc, and other rotary motion valves are not a quarter turn valve.

(iv) Valve type based on types of actuators are used

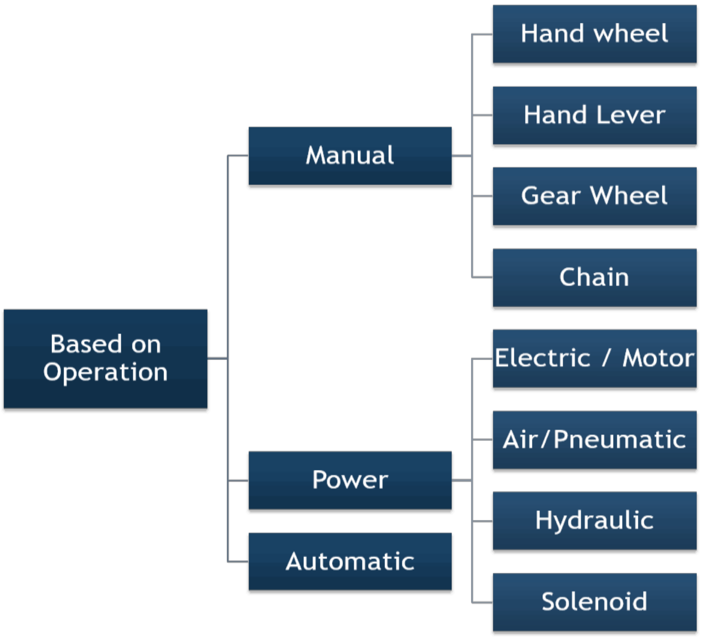


Figure 11.50: Operation Based Valve Type

The valves are classified as per the types of the actuator used to transfer the motion to operate the valve. The valve can be operated manually with the help of a hand wheel, lever, chain, or by a gear wheel. An external power source such as an electric motor, air, hydraulic fluid, or solenoid is used to operate a valve from the control room. The check valve works automatically when subjected to the backflow. Figure 11.50 shows operation based valve type.

11.23.1.2 Sluice or Gate Valves

Sluice valves or gate valves are the normal type of valves used for isolating or scouring. They seal well under high pressures and when fully open, offer little resistance to fluid flow.

IS 14846: 2000 (Reaffirmed Year: 2020) – Sluice Valve for Water Works Purposes (50 to 1200 mm Size) – covers requirement for non-rising stem-type sluice valves used for water supply up to 45 °C and having double flanges.

The sluice valves are designated by nominal pressure (PN) defined as maximum permissible gauge working pressure in MPa for the sizes 50 mm to 600 mm diameter as PN1.0 and PN1.6 and also for sizes from 50 mm to1200 mm diameter as PN1.0. The sluice valves are of nominal sizes 50 mm, 65 mm,80 mm, 100 mm, 125 mm, 150 mm, 200 mm, 250 mm, 300 mm, 350 mm, 400 mm, 450 mm, 500 mm, 600 mm, 700 mm, 800 mm, 900 mm, 1000 mm, 1100 mm, and 1200 mm diameter. The nominal sizes refer to the nominal bore of the waterway.

(A) Types of sluice or gate valves

There are following three ways to classify the gate valve:

(i) Types of discs

a. Solid taper wedge
b. Flexible wedge
c. Split wedge or parallel disc valve

(ii) Types of body bonnet joint

a. Screwed bonnet
b. Bolted bonnet
c. Welded bonnet
d. Pressure seal bonnet

(iii) Types of stem movement

a. Rising stem or OS and Y-Type (outside stem and screw-type)
b. Non-rising stem-type

(i) Classification by Type of Disc

a) Solid wedge gate valve

Solid wedge is the most common and widely used disc-type because of its simplicity and strength. A valve with a solid wedge may be installed in any position, and it is suitable for almost all fluids. It can be used in turbulent flow also.

However, it does not compensate for changes in seat alignment due to pipe loads or thermal expansion. So, this type of disc design is most susceptible to leakage. Solid wedge is subjected to thermal locking if used in high-temperature service.

Thermal locking is a phenomenon in which wedge is stuck between the seats due to the expansion of the metal. Solid wedge gate valves are generally used in moderate to lower pressure temperature applications. Figure 11.51 shows typical sketch of a sluice valve for size 150 mm diameter with thrust plate.

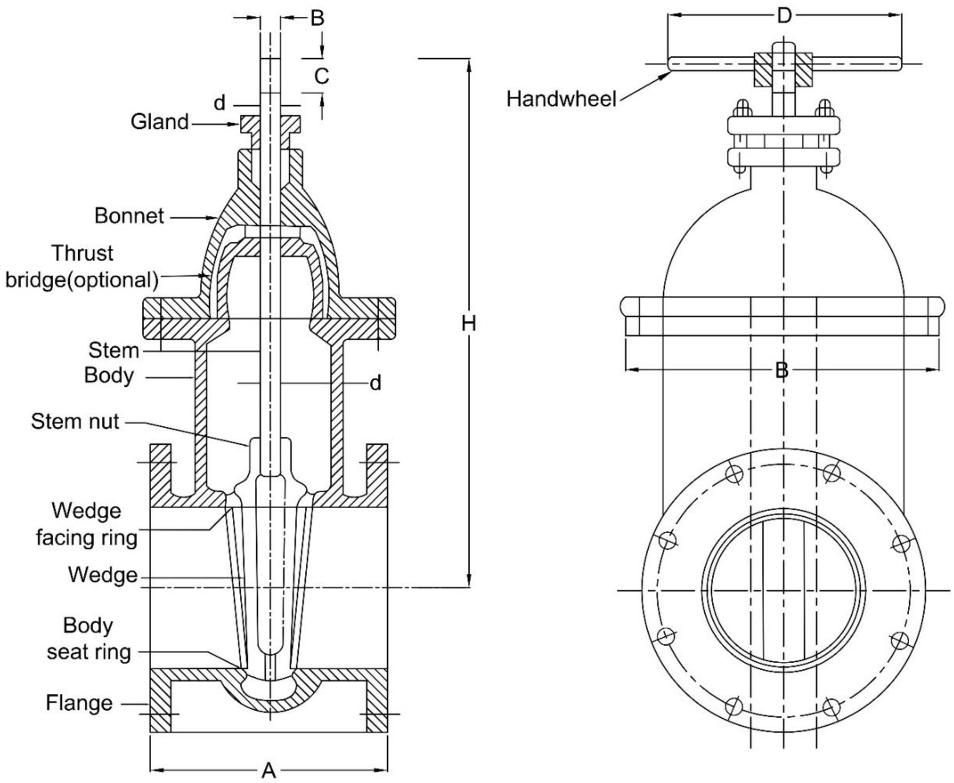


Figure 11.51: Typical sketch of a sluice valve for size 150mm diameter with thrust plate

There are two types of spindles for raising the gate:

a. a rising spindle which is attached to the gate and does not rotate with the hand wheel; and
b. a non-rising spindle which is rotated in a screwed attachment in the gate. The rising spindle is easy to lubricate.

The gate may be parallel sided or wedge shaped. The wedge gate seals best but may be damaged by grit. For low pressure, resilient or gunmetal scaling faces may be used. For high pressure, stainless steel seals are preferred. Sluice valves are not intended to be used for continuous throttling, as erosion of the seats and body cavitation may occur. If small flows are required the bypass, valve is more suitable for this duty. Despite sluice valve's simplicity and positive action, they are sometimes troublesome to operate. They need a big force to unseat them against high unbalanced pressure and large valves take many minutes to turn open or closed, for which power operated or manual operated actuators are also used. Some of these problems can be overcome by installing a valve with a smaller bore than the pipeline diameter.

In special situations, variations of sluice valves suited to the needs are used; needle valves are preferred for fine control of flow, butterfly valves for ease of operation and cone valves for regulating the time of closure and controlling water hammer.

b) Flexible wedge gate valve

The flexible wedge is a one-piece solid disc with a cut around the perimeter. These cuts vary in size, shape, and depth. A shallow, narrow cut on wedge perimeter gives less flexibility but retains strength. A cast-in recess or deeper and wider cut on wedge perimeter gives more flexibility but compromises the strength.

This design improves seat alignment and offers better leak tightness. It also improved performance in situations where thermal binding is possible. Flexible wedges gate valves are used in steam systems.

Thermal expansion of the steam line sometime causes distortion of valve bodies which may lead to thermal blinding. The flexible gate allows the gate to flex as the valve seat compresses due to thermal expansion of the steam pipeline and prevents thermal blinding.

The disadvantage of flexible gates is that line fluid tends to collect in the disc. These may result in corrosion and ultimately weaken the disc. Figure 11.52 shows flexible wedge gate valve.

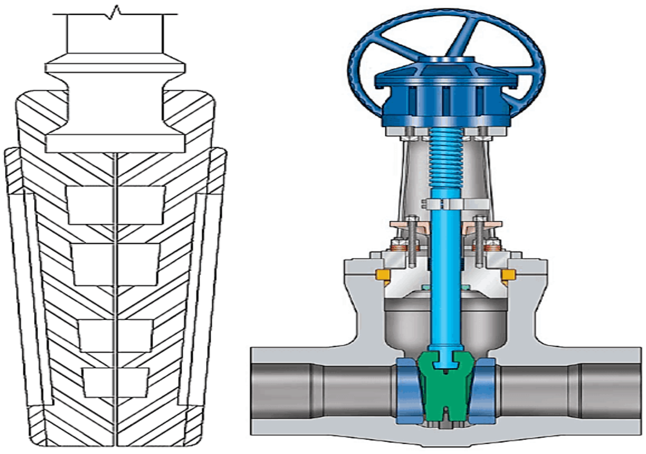


Figure 11.52: Flexible Wedge Gate Valve

c) Split wedge or parallel disc gate valve

Split wedge disc consists of two solid pieces and holds together with the help of a special mechanism. You can see the same in images. In case one-half of the disc is out of alignment, the disc is free to adjust itself to the seating surface. The split disc can be in a wedge shape or a parallel disc-type.

Parallel discs are spring-loaded, so they are always in contact with seats and give bi-directional sealing. The split wedge is suitable for handling noncondensing gasses and liquids at normal and high temperatures.

Freedom of movement of the disc prevents thermal binding even though the valve may have been closed when a line is cold. This means that when a line is heated by fluid and expands, it does not create thermal blinding. Figure 11.53 shows split wedge parallel disc gate valve.

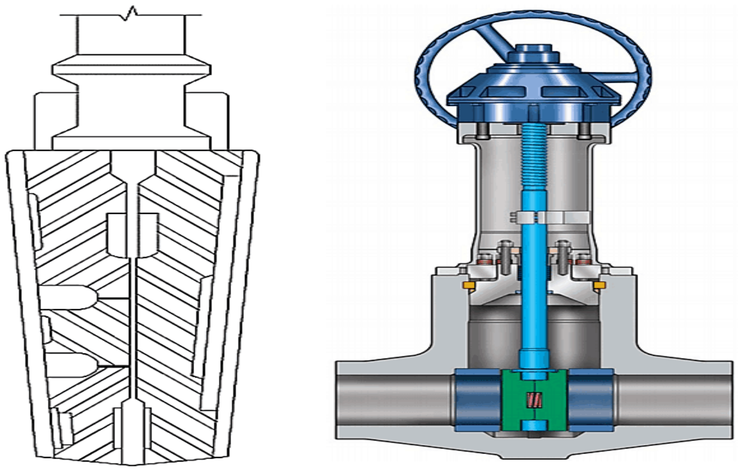


Figure 11.53: Split Wedge parallel discs Gate Valve

(ii) Classification by types of body, bonnet connection

Figure 11.54: Different Valves based on Bonnet Connections

1st is screwed bonnet: This is the simplest design available, and it is used for inexpensive valves.

2nd is bolted bonnet: This is the most popular design and used in a large number of gate valves. This requires a gasket to seal the joint between the body and bonnet.

3rd is welded bonnet: This is a popular design where disassembly is not required. They are lighter in weight than their bolted bonnet counterparts.

4th one is pressure seal bonnet: This type is used extensively for high pressure high-temperature applications. The higher the body cavity pressure, the greater the force on the gasket in a pressure seal valve.

(iii) Classification by types of stem movement

a) OS and Y gate valve or rising stem (outside stem and screw-type)

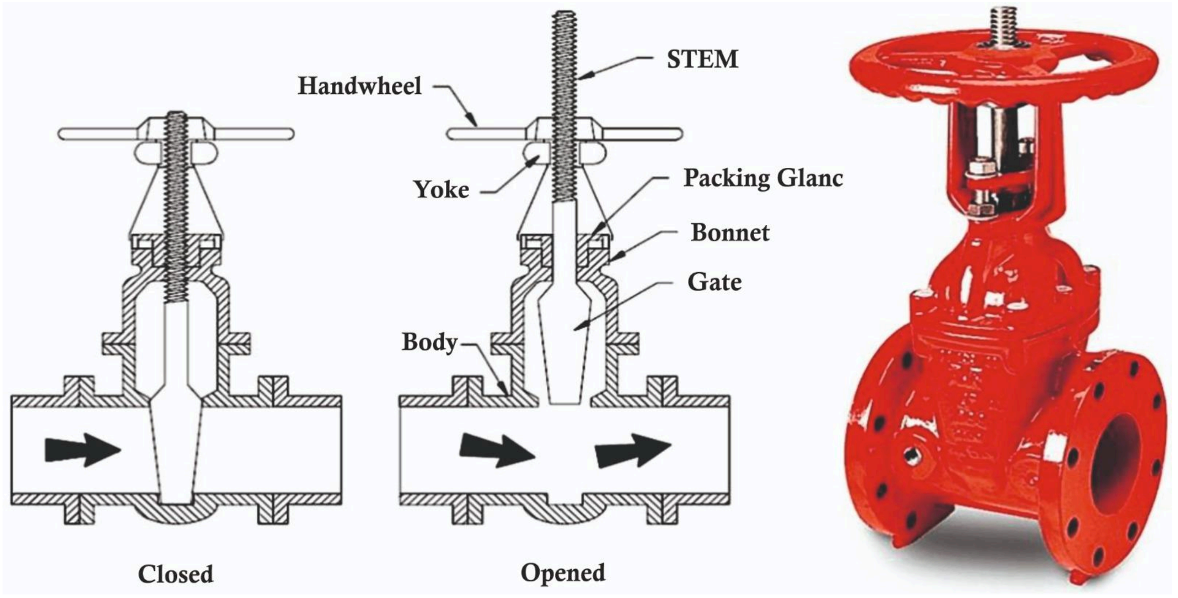


Figure 11.55: OS & Y Gate Valve

For a rising stem valve, the stem will go up while opening the valve and move down when close the valve. Figure 11.55 shows OS and Y gate valve.

Whereas in the case of outside screw design, the only smooth portion is exposed to the flow medium and the stem will rise above the hand wheel. This type of valve is also known as OS and Y valve. OS and Y means outside steam and York.

b) Non-rising stem gate valve or insider screw valve

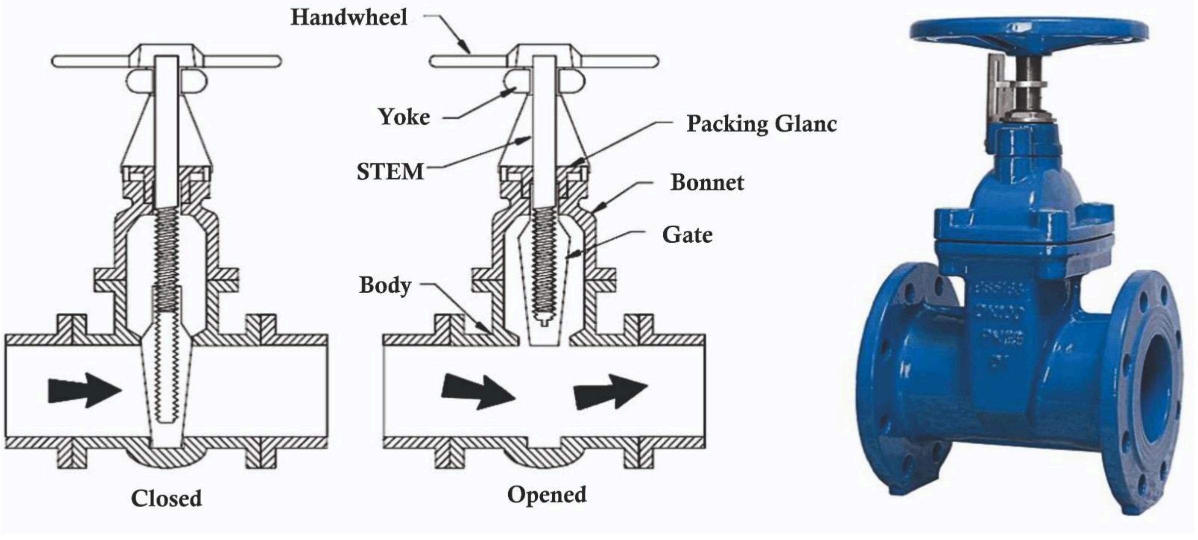


Figure 11.56: Non-rising Stem Gate Valve

There is no upward movement of the stem in a non-rising stem-type. The valve disc is threaded internally. The disc travels along the stem like a nut when the stem is rotated. You can see the image. In this type of valve, stem threads are exposed to the flow medium. Figure 11.56 shows non-rising stem gate valve.

Therefore, this design is used where space is limited to allow linear stem movement, and the flow medium does not cause erosion, corrosion, or wear and tear to stem material. This type of valve also known as an insider screw valve.

(B) Gate valve applications

- Gate valves are used in almost all fluid services such as air, fuel gas, feedwater, steam, lube oil, hydrocarbon, and all most any services.
- Some special gate valve is used in slurry and powder product also such as knife gate valve.

(C) Advantages and disadvantages of sluice valve

Advantages	Disadvantages
Pressure drop during operation is very less.	A gate valve is slow in operation. Opening and closing take time which is good also as it reduces the chance of hammering.
Most of the gate valve can be used as bi-directional.	When partially open it creates vibration and noise.
Most suitable for high pressure and temperature application and required less maintenance.	Repairs, such as lapping and grinding of seats are more difficult due to limited access.
---	It cannot be used to control the flow.

11.23.1.3 Butterfly Valves

IS 13095: 2020 – Butterfly Valves for General Purposes (First Revision) covers double flanged and wafer-type of metal-seated, resilient-seated, cast iron, ductile iron, and carbon steel and lined butterfly valves for general purposes. Valves covered under tis standard can be operated manually, pneumatically, hydraulically, or electrically.

Butterfly valves are used to regulate and stop the flow especially in large size conduits. They are sometimes cheaper than sluice valves for larger sizes and occupy less space. Butterfly valves with no sliding parts have the advantages of ease of operation, compact size, reduced chamber or valve house and improved closing and retarding characteristics.

(A) Valves applications:

a. Tight shut-off – A valve having no visible leakage past the disc in closed position under test condition;
b. Regulating – A valve intended for regulating purposes and which may have a clearance between the disc and the body in closed position;
c. Low leakage – A valve which has specified maximum leakage rate past the disc in the closed position.
d. A butterfly valve is used in many different fluid services, and they perform well in slurry applications also. They can be used in liquids, steam, cryogenics, cooling water, air, gasses, firefighting, and vacuum services.
e. Butterfly valve is used in all type of industries application even in high pressure and temperature services.

These would involve slightly higher head loss than sluice valves and also are not suitable for continuous throttling. The sealing is sometimes not as effective as for sluice valves especially at high pressures. They also offer a fairly high resistance to flow even in fully open state, because the thickness of the disc obstructs the flow even when it is rotated to fully open position. Butterfly valves as well as sluice valves are not suited for operation in partly open positions as the gates and seatings would erode rapidly. Both types require high torques to open them against high pressure, they often have geared hand wheels or power-driven actuators.

Butterfly valves with loose sealing ring are sometimes not effective, especially at higher pressures. Butterfly valves with fixed liner can overcome this shortcoming. Furthermore, the butterfly valves with fixed liner needs no frequent maintenance for replacement of sealing ring as in the case of butterfly valves with loose sealing ring. The fixed liner design butterfly valves are now available in India suitable for working pressures up to 16 kg/sq. cm. Presently, there is no IS for the fixed liner butterfly valves.

(B) Butterfly valve types

A butterfly valve is a quarter turn rotary motion valve that is used to stop, regulate, and start the flow. Butterfly valves are a quick open type. A 90° rotation of the handle can completely close or open the valve. Normally, they are used in systems where a positive shut-off is not required and are of the following types:

i. Wafer-type
ii. Lug style-type
iii. Flanged-type
iv. Butt welded end types
v. Zero offset
vi. Double offset
vii. Triple offset

Large butterfly valves are usually equipped with gearbox-type actuator, where the handwheel is connected to the stem via a gearbox. This will reduce the force but at the same time reduce the speed of the operation. This type of valve should be installed in the open position. If the valve is closed during installation, the rubber seat will wedge against the valve disc and make it difficult to open.

Butterfly valve types based on body construction:

Based on the type of ends of the body butterfly valves are available in following types.

(i) Both flanged ends
(ii) Wafer-type ends
(iii) Lug-type ends
(iv) Butt welded-type ends

(i) Wafer types

The wafer body is placed between pipe flanges, and the flange bolts surround the valve body. A wafer type butterfly valve is easy to install but it cannot be used as an isolation valve. Figure 11.63 shows a wafer type butterfly valve.

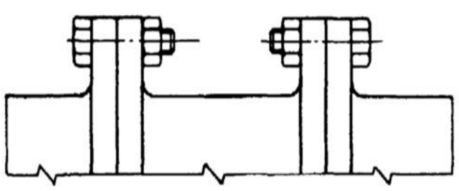


Figure 11.57: Butterfly Valve, Double Flanged Type

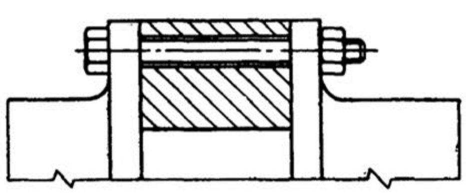


Figure 11.58: Butterfly Valve, Single Flange Wafer Type

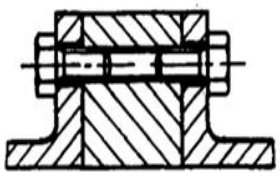


Figure 11.59: Wafer Lugged Screw

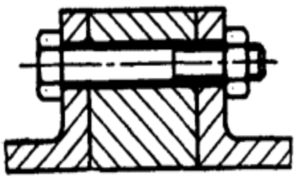


Figure 11.60: Wafer Lugged through bolt

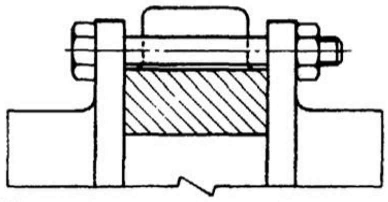


Figure 11.61: Butterfly Valve, Flangeless Wafer Type

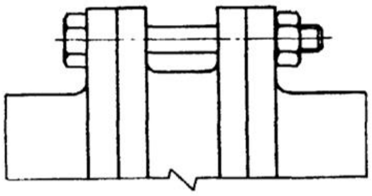


Figure 11.62: Butterfly Valve, U-Section Wafer Type

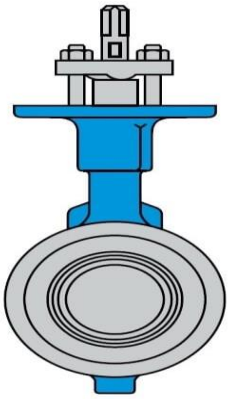


Figure 11.63: Wafer Type Butterfly Valve

(ii) Lug style

The lug body has protruding lugs in the periphery of a body that provides passage to bolt holes that match with those in the flanges. Figure 11.64 shows a lug type butterfly valve.

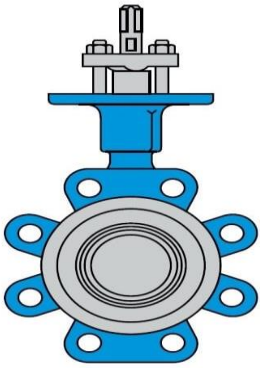


Figure 11.64: Lug Type Butterfly Valve

(iii) Flanged type

In these types, the body has flanged that match with pipe flange dimension. Figure 11.57 shows butterfly valve, double flanged type, Figure 11.58 shows butterfly valve, single flange wafer type, and Figure 11.65 shows double flanged butterfly valve.

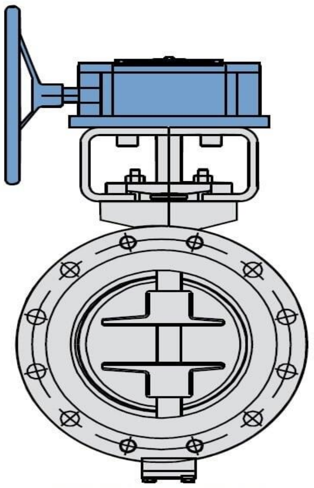


Figure 11.65: Double Flanged Butterfly Valve

(iv) Butt welded end types

These types of ends are used in high-pressure services and it directly welded to the pipe. Figure 11.66 shows butt welded end type.

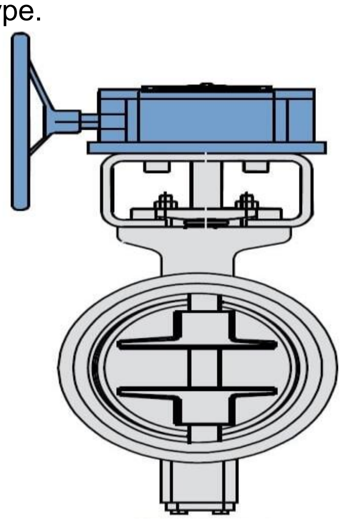


Figure 11.66: Butt Welded End Type

(v) Zero offset butterfly valve

Zero offset design used for the valve that used in low pressure and temperature services. In this design, the disc and shaft axis are concentric with the valve body. In the open position, the disc divides the flow into two equal halves, with the disc in the middle and parallel to the flow.

This type of valve has a resilient seat. Sealing is achieved when the disc deforms the soft seat. There is friction between the disc and seat during the full operating cycle which is the disadvantage of zero offset valve. Figure 11.67 shows zero offset butterfly valve.

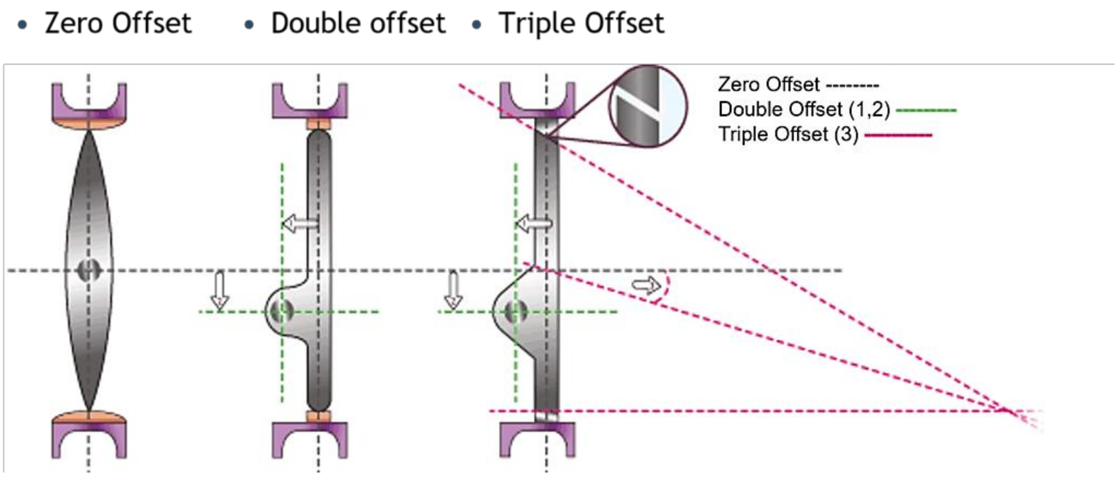


Figure 11.67: Zero offset Butterfly Valve

(vi) Double Offset Butterfly Valve

In double offset, the disc is offset from the valve centre line and also from valve body centre line. You can see this in the image where one and two are written. This creates a cam action during operation that lifts the seat out of the seal.

Double offset makes opening and closing smooth as friction is applicable only during the first few degrees of opening and final few degrees of closing, approx. 10° of opening and closing. Figure 11.68 shows double offset butterfly valve.

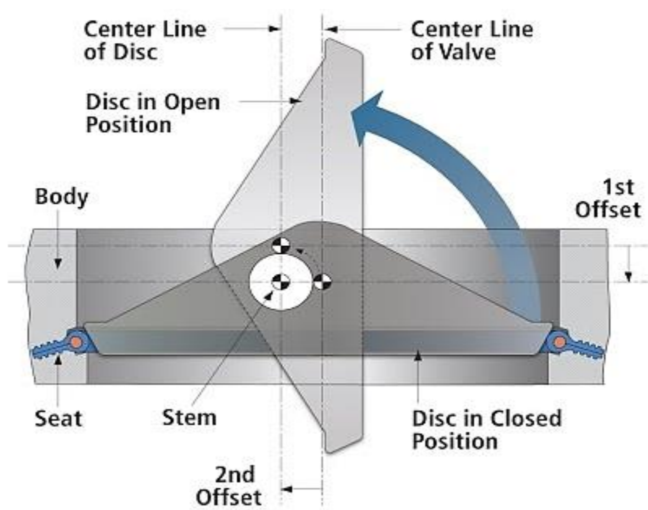


Figure 11.68: Double Offset Butterfly Valve

(vii)Triple offset butterfly valve

In triple offset design, the third offset created by the geometry design of the seating surface. The seat is machined into an offset conical profile resulting in a right-angled cone.

This ensures frictionless stroking throughout its operating cycle. Contact is only made at the final point of closure with the 90° angle acting as a mechanical stop; the metal-seated valve uses triple offset design. Figure 11.69 shows butterfly valve body parts and Figure 11.70 shows the typical butterfly valve has a short circular body, a round disc, shaft, and metal or soft seats.

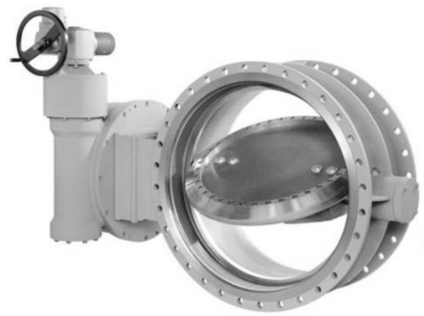


Figure 11.69: Butterfly Valve Body Parts

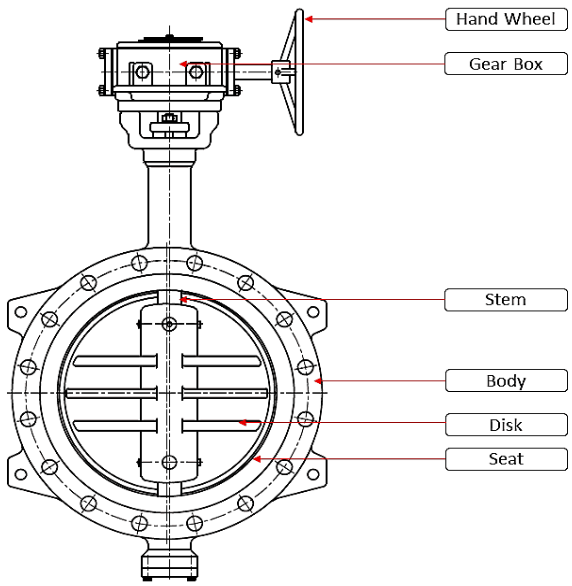


Figure 11.70: The typical butterfly valve has a short circular body, a round disc, shaft, and metal or soft seats

(viii) Seat types

Butterfly valves can be metal-to-metal seated, soft seated, or with a fully lined body and disc. The first image is of a soft seated fully lined body and disc valve. The second image is a soft seat with metal disc and the third image is a metal-to-metal seat type valve.

The disc of butterfly valve can be concentric or eccentric with the valve body.

(C) Advantages and disadvantage

Advantages	Disadvantages
Butterfly valve is suitable for large valve applications due to compact, lightweight design that requires considerably less space, as compared to other valves	Throttling is limited to low differential pressure services and that too with a 30- to 80-degree disc opening.
Due to a quick operation, it needs less time to open or close	There is a chance for cavitation and choke as the disc is always in the flow Turbulence flow can affect the disc movement.
The maintenance costs are usually low compared to other valve types	
A pressure drop across a butterfly valve is small	

Advantages	Disadvantages
Valve with non-metallic seating can be used in chemical or corrosive media	

11.23.1.4 Globe Valves

Globe valves have a circular seal connected axially to a vertical spindle and hand wheel. The seating is a ring perpendicular to the pipe axis. The flow changes direction through 90° twice thus resulting in high head losses. These valves are normally used in small bore pipe work and as taps, although a variation is used as a control valve.

A globe valve is a linear motion valve used to stop, start, and regulate the fluid flow. The globe valve disc can be removed entirely from the flow path, or it can completely close the flow path. During the opening and closing of the valve, the disc moves perpendicularly to the seat.

This movement creates the annular space between the disc and seat ring that gradually closes as the valve closed. This characteristic provides the globe valve good throttling ability required for regulating the flow.

Leakage from the globe valve seat is less as compared to the gate valve, mainly due to right angle contact between the disc and seat ring, which allows tighter seal between seat the disc.

IS 13114: 1991 (Reaffirmed Year: 2022) – forged brass gate, globe and check valves for water works purposes refers. In the below globe valve diagram, you can see how the globe valve functions. The image also shows flow direction.

Globe Valve Diagram

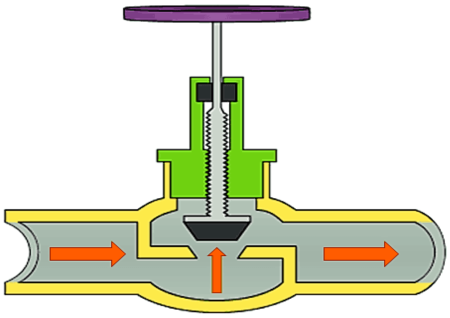


Figure 11.71: Disc Close against the Flow Direction

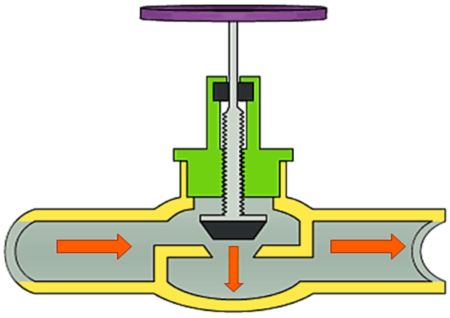


Figure 11.72: Disc Close in Flow Direction

Globe valves can be arranged in such a way that the disc closes against the flow or in the same direction of flow. Figure 11.71 shows disc close against the flow direction and Figure 11.72 shows disc close in flow direction.

When the disc closes in the direction of flow, the kinetic energy of the fluid helps closing but obstructs the opening. This characteristic is preferable when a quick-acting stop is required.

When the disc closes against the direction of flow, the kinetic energy of the fluid obstructs closing but helps to open the valve. This characteristic is preferable when quick-acting start is required. Figure 11.73 is a sketch showing globe valve parts such as body, bonnet, stem, seat, disc, etc.

Sr No	Globe Valve
1	Body
2	Bonnet
3	Disc
4	Disc nut
5	Body seat ring
6	Stem
7	Back seat
8	Gland follower
9	Gland flange
10	Yoke nut
11	Handwheel
12	Handwheel nut
13	Stud bolts
14	nuts
15	Stud bolts
16	nuts
17	Bonnet gasket
18	Packing

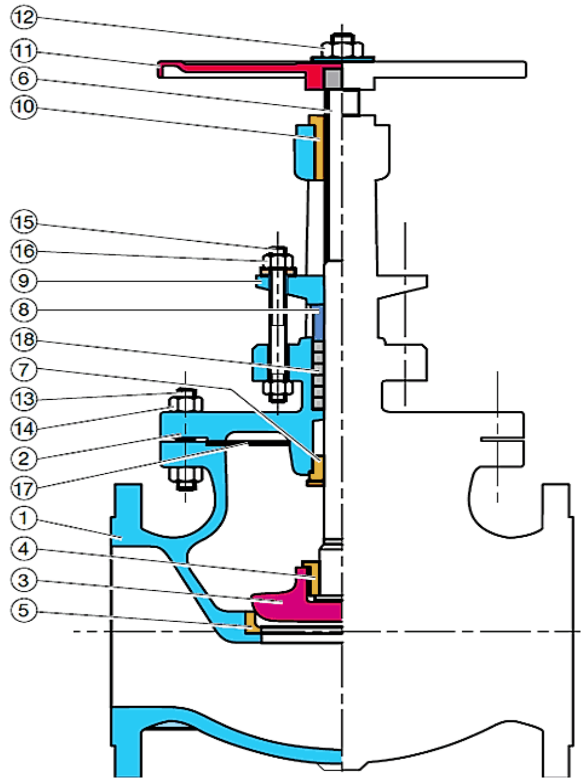


Figure 11.73: Sketch showing globe valve parts such as Body, Bonnet, Stem, Seat, Disc, etc.

(A) Types of globe valve based on disc types

Globe valve is available in many different types of disc arrangement. The most used disc designs are listed below.

1. Ball-type
2. Needle-type
3. Composite-type

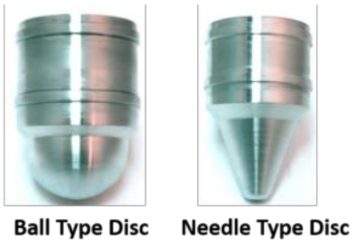


Figure 11.74: Ball-Type Disc and Needle-Type Disc

The ball disc design is used in low pressure and low-temperature systems. It is capable of throttling flow, but in principle, it is used to stop and start the flow.

Needle disc design provides better throttling as compared to ball or composition disc design. A wide verity of long and tapered plug discs is available to suit different flow conditions.

Composition disc is used to achieve better shutoff. A hard, non-metallic insert ring is used in composition disc design.

Figure 11.74 shows ball-type disc and needle-type disc.

(B) Types of globe valve based on body type

Depending on the type of body, there are three types of globe valves;

(i) Z types
(ii) Y types
(iii) Angle types

(i) Z types globe valve

The simplest design and most common type are a Z-body. The Z-shaped partition inside the globular body contains the seat. The horizontal seating arrangement of the seat allows the stem and disc to travel at a perpendicular to the pipe axis resulting in a very high-pressure loss.

The valve seat is easily accessible through the bonnet which is attached to a large opening at the top of the valve body. Stem passes through the bonnet like a gate valve.

This design simplifies manufacturing, installation, and repair. This type of valve is used where pressure drop is not a concern and throttling are required. Figure 11.75 shows Z types globe valve.

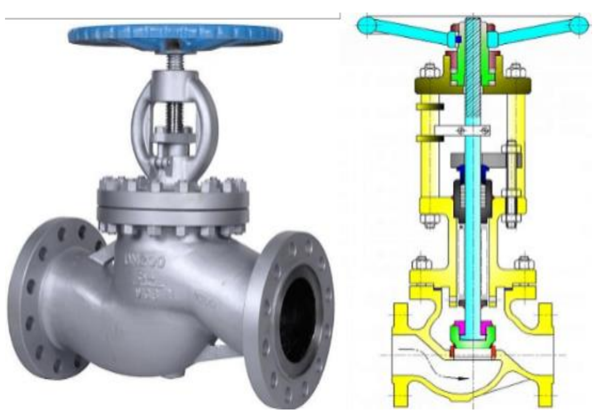


Figure 11.75: Z-types Globe Valve

(ii) Y types globe valve

The Y-type design is a solution for the high-pressure drop problem in Z-type valves. In this type, seat and stem are angled at approximately 45° to the pipe axis. Y-body valves are used in high pressure and other critical services where pressure drop is concerned. Figure 11.76 shows Y types globe valve.

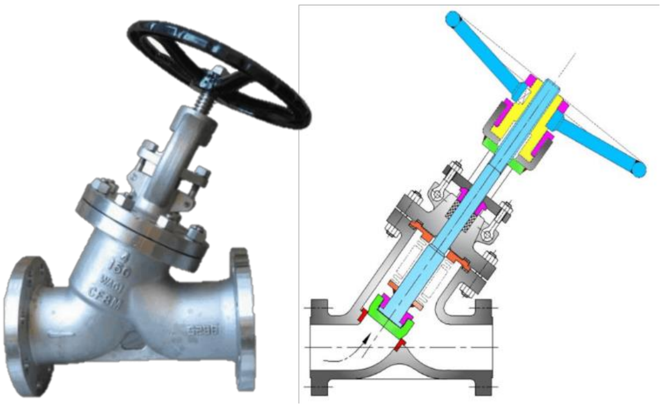


Figure 11.76: Y Types Globe Valve

(iii) Angle types globe valve

Angle globe valve turns the flow direction by 90º without using an elbow and one extra pipe weld. Disc open against the flow. This type of globe valve can be used in the fluctuating flow condition also, as they are capable of handling the slugging effect. Figure 11.77 shows angle types globe valve.

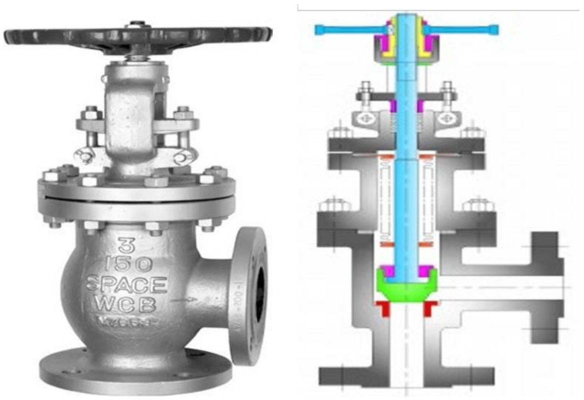


Figure 11.77: Angle types Globe Valve

(C) Types of Globe Valve Based on Body Bonnet Connection

a) **Screwed bonnet:** This is the simplest design available and it is used for inexpensive valves.
b) **Bolted bonnet:** This is the most popular design and used in a large number of globe valves. This requires a gasket to seal the joint between the body and bonnet.
c) **Welded Bonnet:** This is a popular design where disassembly is not required. They are lighter in weight than their bolted bonnet counterparts.
d) **Pressure Seal Bonnet:** This type is used extensively for high pressure high-temperature applications. The higher the body cavity pressure, the greater the force on the gasket in a pressure seal valve.

(D) Application of globe valve

Globe valves are used in the systems where flow control is required and leak tightness is also important.

- It used in high point vents and low point drains when leak tightness and safety are major concerns. Otherwise, you can use a gate valve for drain and vent.
- It can be used in feedwater, chemical, air, lube oil, and almost all services where pressure drop is not an issue.

- This valve is also used as an automatic control valve, but in that case, the stem of the valve is a smooth stem rather than threaded and is opened and closed by lifting action of an actuator assembly.

(E) Advantages and disadvantage of globe valve

Advantages	Disadvantages
Better shut off as compared to gate valve	High head loss from two or more right-angle turns of flowing fluid within the valve body
Good for frequent operation as no fear of wear of seat and disc	Obstructions and discontinuities in the flow path lead to a high head loss
Easy to repair, as seat and disc can be accessed from the valve top	In a large high-pressure line, pulsations and impacts can damage internal trim parts
Fast operation compares to gate valve due to shorter stroke length	A large valve requires considerable power to open and create noise while in operation
Usually operated by an automatic actuator	It is heavier than other valves of the same pressure rating
-----	Costlier compared to the gate valve

11.23.1.5 Needle and Cone Valves

Needle valves are more expensive than sluice and butterfly valves but are well suited for throttling flow. They have a gradual throttling action as they close, whereas sluice valves and butterfly valves offer little flow resistance until practically shut and may suffer cavitation damage.

A needle valve is a manual valve that used where continuous throttling of flow is required for regulation. Needle valves are similar to the globe valve in design with the biggest difference is the sharp needle-like disc.

Needle valves are designed to give very accurate control of flow in small diameter piping systems. They get their name from their sharp-pointed conical disc and matching seat. Figure 11.78 shows needle and cone valves.

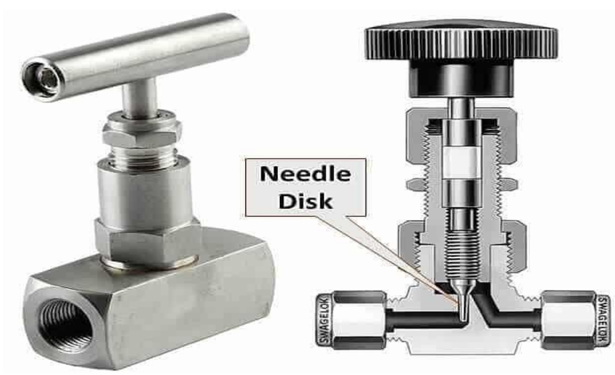


Figure 11.78: Needle and Cone Valves

Fluid flowing through the valve turns 90º and passes through an orifice. Due to needle shape disc, a certain portion of the disc will pass through seat opening before disc comes in contact with the seat, which has matching tapered design as a disc. This arrangement permits a very gradual increase or decrease in the size of the opening.

Needle valve has forged and machined body. This body can be of forged carbon steel or stainless steel depending on the requirements of the services. A seat can be a soft, metal, or composite, same as a globe valve. Normally, needle valves are used in smaller sizes and are provided with either screwed or socket weld end.

All the aspects of needle valve are the same as a global valve except its size and pointed needle-like disc. You can refer to a globe valve for more detail.

(A) Needle valve application

- All field analogue instruments are fitted with a needle valve to control flow entry, where sudden surges of fluid under pressure can damage the instruments.
- Needle valves can be used in situations where the flow must stop gradually and, in the application, where precise adjustments of flow are required or where a small flow rate is desired such as sample points in the piping.
- Needle valves can be used as both on/off and throttle valves.
- Used in all types of industries for controlling and metering applications of fluid such as steam, air, gas, oil, water, or other non-viscous liquids.

Needle valves may be used with counterbalance weights, springs, or actuators to maintain constant pressure conditions either upstream or downstream of the valve or to maintain a constant flow. They are resistant to wear even at high flow velocities. The method of sealing is to push an axial needle or spear shaped cone into a seat. There is often a pilot needle which operates first to balance the heads before opening. The cone valve is a variation of the needle valve, but the sealing cone rotates away from the pipe axis instead of being withdrawn axially.

The needle and cone valves are not commonly used in water supply but are occasionally used as water hammer release valves when coupled to an electric or hydraulic actuator.

11.23.1.6 Air Valves

When a pipeline is filled, air could be trapped at peaks along the profile thereby increasing head losses and reducing the capacity of the pipeline. It is also undesirable to have air pockets in the pipe as they may cause water hammer pressure fluctuations during operation of the pipeline. Other problems due to air include corrosion, reduced pump efficiency malfunctioning of valves or vibrations. Air valves are fitted to release the air automatically when a pipeline is being filled and also to permit air to enter the pipeline when it is being emptied. Additionally, air valves have also to release any entrained air, which might be accumulated at high points in the pipeline during normal operations.

Without air valves, vacuum may occur at peaks and the pipe could collapse or it may not be possible to drain the pipeline completely.

Air valves require care in selection and even more care in siting and it is good practice to plan the pipeline alignment to avoid air troubles altogether. A special study of the possible air problems is necessary at the design stage itself and provision should be made for suitable corrective measures rather than positioning arbitrary air valves at pipeline peaks.

Locations of air valves can be at both sides of gates at summits, the downstream side of other gates and changes in grade to steeper slopes in sections of line not otherwise protected by air valves.

The valve usually takes the form of a rigid buoyant vulcanite or rubber-covered ball seated on a rubber or metal ring. The sealing element, i.e., the ball, is slated against an opening at the top of the valve when the pipe is full and seals the opening. When the pressure inside the pipe falls below external pressure, the ball drops thereby permitting air to be drawn into the pipe. The valves are mainly available in two forms, either single ball or double ball. The single ball-type can have either a large orifice or a small orifice, the former being only suitable for emptying and filling of pipelines, and the latter for discharging small quantities of entrained air. Double air valves are available which can be classified as dual purpose with a large orifice and small orifice in one unit, with a common connection to the main. For large aqueduct pipelines, a triple orifice air valve is available with two large orifices and one small. For high pressures, stainless steel floats are used instead of the vulcanite-covered balls.

Special designs of air valves are also available which operate satisfactorily with high velocity air discharges. If normal air valves are used under these conditions, there is a danger that the ball might be carried on to its seat by the air stream before the accumulated air has been fully released.

Air valves can be provided with an integral stop valve or alternatively, and preferably, a standard sluice valve can be bolted to the inlet flange, which must be of adequate size for its duty. Regular maintenance checks on at least an annual basis should be carried out to ensure that the balls are free to move and that the seals do not leak. If an air valve is isolated for any reason in very cold weather, the body should be drained to prevent frost damage; a plug cock can be fitted at the base of the body for this purpose. Trapped chamber drainage is essential to prevent any possibility of stagnant of polluted water or air entering the pipeline.

Automatic air valves in urban streets present a serious contamination risk, since they must have air vents that could, in some circumstances, admit polluted surface water. Constructing an air valve chamber as watertight as possible and fitting a ball valve interceptor as an outlet to a storm water sewer is a practice to obviate this possibility. Using manually operated air valves in the streets, it being the routine duty of a turncock in the area to air the main, to minimise the risk of serious contamination, is yet another practice.

The following ratios of air valves to conduit diameter provide common but rough estimates of needed sizes:

For release of air only	1:12
For admission as well as release of air	1:8

An analysis of air inlet valves for steel pipelines, Parmakian takes the compressibility of air into account and combines equations for safe differential pressures of cylindrical steel pipe, pipe flow, and air flow, in the following approximate relationships:

$$\frac{d_a}{d} = 1.99 \times 10^{-2}\sqrt{\frac{\Delta V}{C}}\left[1 - \frac{P_1}{P_2} \times 0.288\right] - 0.25 \qquad (11.4)$$

For P_2 > 0.53 P_1 , and as

$$\frac{d_a}{d} = 3.91 \times 10^{-2}\sqrt{\frac{\Delta V}{C}}\left(\frac{P_2}{P_1}\right)^{0.356} \qquad (11.5)$$

For $P_2 \leq 0.53\, P_1$ because air flow cannot increase beyond a critical differential of 0.488 Kg/cm^2.

In these equations, d_a and d, respectively, are the diameters of the air orifice and pipe, ΔV is the difference in the velocities of flow on each side of the inlet valve, C is the coefficient of discharge of the valve, P_2 and P_1, are the pressures inside and outside the pipe respectively, with $P_1 - P_2$ not exceeding one-half of the collapsing pressure as a matter of safety.

The equations apply strictly only to elevations of 304.8 m above mean sea level at 40 degrees latitude (g = 9.81 mps) temperatures of 25.32 °C, 20% humidity, an adiabatic expansion for which $pv^n = pv^{1.40}$, the air occupying a volume of 0.87 cum/Kg.

(A) Air Release Valves

Air release valves are designed specifically to vent, automatically and when necessary, air accumulations from lines in which water is flowing. Such accumulations of air tend to collect at high points in the pipeline. Air which accumulates at such peaks, reduces the useful cross-sectional area of the pipe, and therefore induces a friction head factor that lowers the pumping capacity of the entire line. The use of air release valves eliminates the possibility of this air binding and permits the flow of water without damage to pipeline. Releasing small burst of air at frequent intervals with the pipeline flowing full, the valve is immensely helpful in purging small bursts of accumulated air in the pipeline thereby enhancing discharge capacity of water mains. The float configuration, the lever arrangement, and the orifice size are decided upon after careful study of the site condition. By changing the orifice size, it can tackle large differential pressures (0.5 to 16 bar). Figure 11.79 shows air release valves.

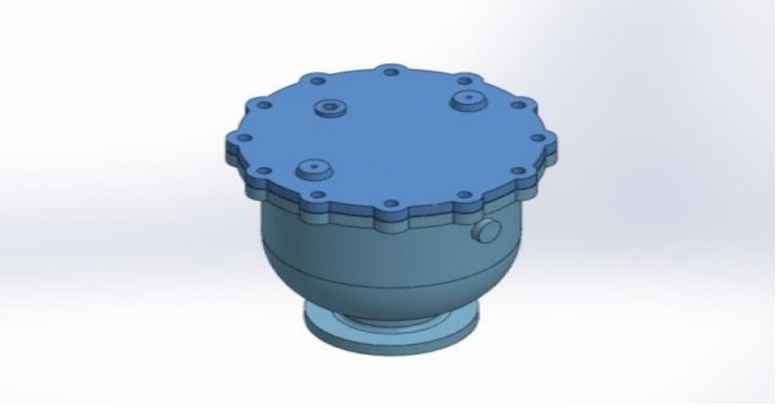


Figure 11.79: Air Release Valves

Small orifice air valves are designated by their inlet connection size, usually 12 to 50 mm diameter. This has nothing to do with the air release orifice size which may be from 1 to 10 mm diameter. The larger the pressure in the pipeline, the smaller need be the orifice size. The volume of air to be released will be a function of the air entrained which is on the average 2% of the volume of water (at atmospheric pressure).

The small orifice release valves are sealed by a floating ball, or needle which is attached to a float. When a certain amount of air has accumulated in the connection on top of the pipe, the ball will drop or the needle valve will open and release the air. Small orifice release valves are often combined with large orifice air vent valves on a common connection on top of the pipe. The arrangement is called a double air valve. An isolating sluice valve is normally fitted between the pipe and the air valves.

Double air valves should be installed at peaks in the pipeline, both with respect to the horizontal and the maximum hydraulic gradient. They should also be installed at the ends and intermediate points along a length of pipeline which is parallel to the hydraulic grade line. It should be borne in mind that air may be dragged along in the direction of flow in the pipeline and may even accumulate in sections falling slowly in relation to the hydraulic gradient. Double air valves should be fitted every ½ to 1 km along descending sections, especially at points where the pipe dips steeply.

Air release valves should also be installed all along ascending lengths of pipeline where air is likely to be released from solution due to the lowering of the pressure, again especially at points of decrease in gradient. Other places where air valves are required are on the discharge side of pumps and at high points on large mains and upstream of orifice plates and reducing tapers.

Air relief towers are provided at the first summit of the line to remove air that is mechanically entrained as water is drawn into the entrance of the pipeline.

(B) Air Inlet Valves

In the design and operation of large steel pipelines, where gravity flow occurs, considerations must be given to the possibility of collapse in case the internal pressure is reduced below that of atmosphere. Should a break occur in the line at the lower end of a slope, a vacuum will in all probability be formed at some point upstream from the break due to the sudden rush of water from the line. To prevent the pipe from collapsing, air inlet (vacuum breaking) valves are used at critical points.

These valves normally held shut by water pressure, automatically open when this pressure is reduced to slightly below atmosphere, permitting large quantities of air to enter the pipe, thus effectively preventing the formation of any vacuum. In addition to offering positive protection against extensive damage to large pipelines, by prevention of vacuum, they also facilitate the initial filling of the line by the expulsion of air wherever the valves are installed.

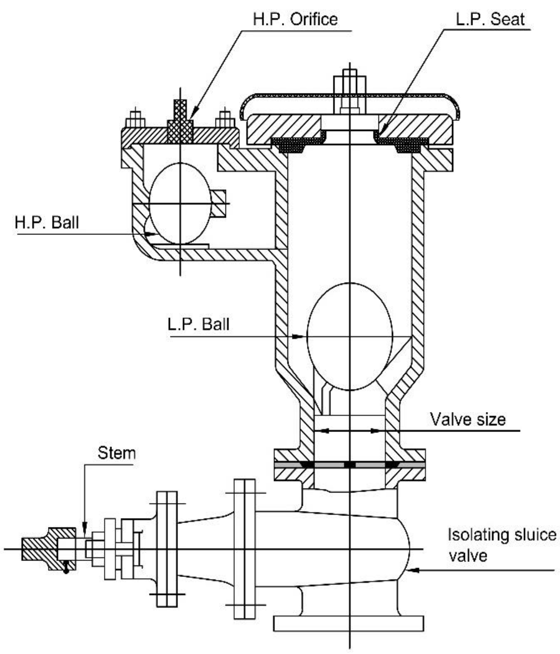
Air inlet valves should be installed at peaks in the pipeline, both relative to the horizontal and relative to the hydraulic gradient. Various possible hydraulic gradients, including reverse gradients during scouring, should be considered. They are normally fitted in combination with an air release valve.

Often, air release valves are used in conjunction with them, the purpose of these being to vent air accumulations that may occur at the peaks after the line has been put into operation.

(C) Kinetic Air Valves

In case of ordinary air valve, single orifice (small or large) type, the air or water from the rising main is admitted in the ball chamber of the air valve from one side of the ball. The disadvantages with this type of valve are that (a) once the ball goes up, it does not come down even when air accumulates in the ball chamber and (b) due to air rushing in, it stirs the ball making it stick to the upper opening which does not fall down unless the pressure in the main drops. The kinetic air valve overcomes these deficiencies since the air or water enters from the bottom side of the ball and the air rushing around ball exerts the pressure and loosens the contact with the top opening and allows the ball to drop down. Figure 11.80 shows double chamber kinetic air valve (onsite), Figure 11.81 shows double chamber kinetic air valve (schematic), Figure 11.82 shows single chamber kinetic air valve (large orifice), and Figure 11.83 shows single chamber kinetic air valve (small orifice).

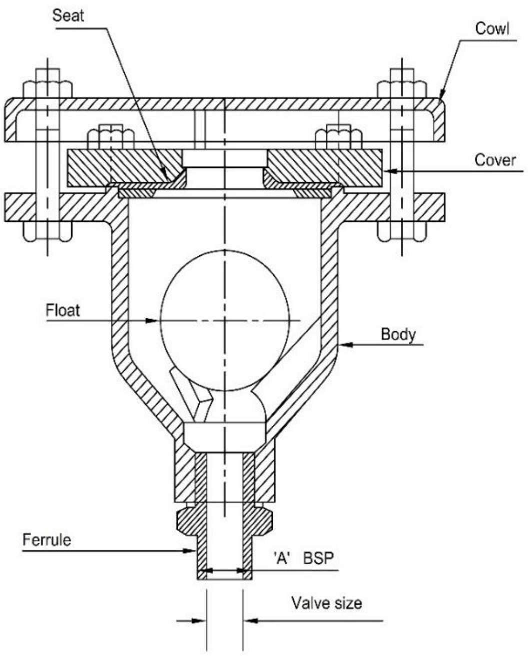
Figure 11.80: Double Chamber Kinetic Air Valve (Onsite)



Major component	Material of construction
Body/Cover/Coal	Cast iron/Ductile iron
L.P./H.P. ball	Vulconite/Rubber covered timber ball
L.P. Seat	Elastomer
H.P. Orifice	Gun metal
Isolating sluice valve	DF Metal seated
Isolating sluice valve stem	Stainless steel: AISI 410/AISI 431/AISI 304/AISI 316

Valve size	50Ø	80Ø	100Ø	150Ø	200Ø

Figure 11.81: Double Chamber Kinetic Air Valve (Schematic)

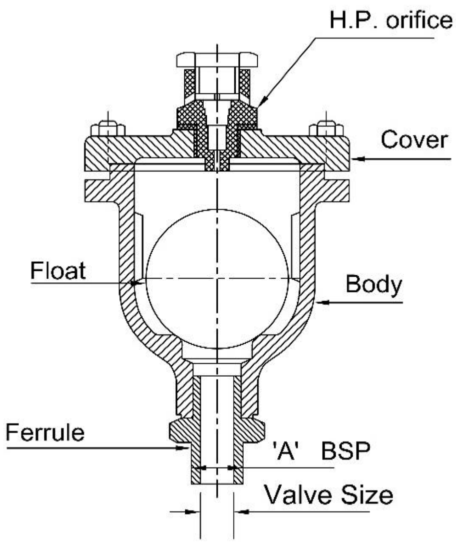


Major component	Material of construction
Body/Cover/Coal	Cast iron
Floot	Vulcanite covered timber ball
Seat	Elastomer
Ferrule	Standard

Valve size	25Ø	50Ø
'A' BSP	1"	2"

Figure 11.82: Single Chamber Kinetic Air Valve (Large Orifice)

Single air valve (small orifice)



Major component	Material of construction
Body/Cover	Cast iron
Floot	Rubber covered timber ball
Orifice	Gun metal
Ferrule	Standard

Valve size	25Ø
'A' BSP	1"

Figure 11.83: Single Chamber Kinetic Air Valve (Small Orifice)

Pipelines must breathe, inhale air when being emptied (to prevent collapse), or exhale when being charged with water. And while the pipeline is running full, the dissolved air in water must be purged from time to time to ensure optimum health of the system.

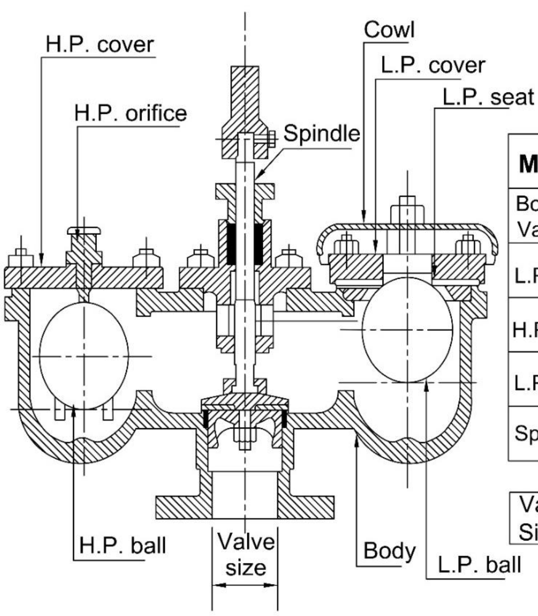
There are four principal types available besides the two small sizes for smaller mains, each having its own positive features. An isolating valve must essentially be provided below the air valve to isolate it for carrying out maintenance in the air valve, with the pipeline below running full.

(D) Guideline for Usage

- For mains size larger than 1200 NB, it is recommended that a cluster of 2 or 3 be used in one location.
- Normally 10 bar rated, 16 bar or higher pressure on request.
- Single air valve of size 1″ as per IS: 14845 can be supplied, suitable for 100 NB mains.
- Other special valves for water application are air vacuum valve (up to 350 NB) and air release valve (150 NB). Table 11.4 below shows valve sizes for different mains sizes. Figure 11.84 shows double air valve with screwed down isolating valve, Figure 11.85 shows single chamber triple function air valve, and Figure 11.86 shows combination of air valve (tamper proof).

Table 11.4: Valve Sizes for Different Mains Sizes

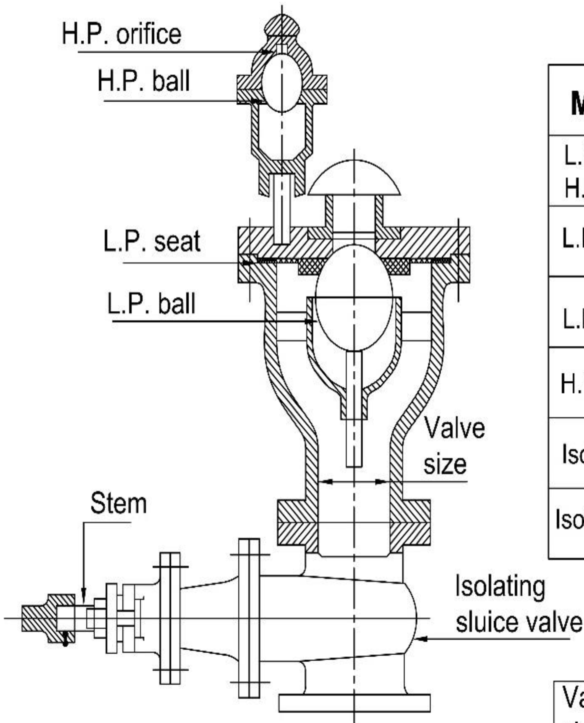
Valve size	For Mains size
50ø	Up to 200ø
80ø	225–350ø
100ø	400–500ø
150ø	600–900ø
200ø	1000–1200ø



Major component	Material of construction
Body/L.P.& H.P.cover/ Valve/Cowl	Cast Iron/Ductile Iron
L.P./Seat ring	Elastomer
H.P. Orifice	Gun metal
L.P. & H.P. ball	Vulconite/Rubber covered timber ball
Spindle	Stainless steel: AISI 410/AISI 304/AISI 316

Valve Size	50Ø	80Ø	100Ø	125Ø	150Ø	200Ø

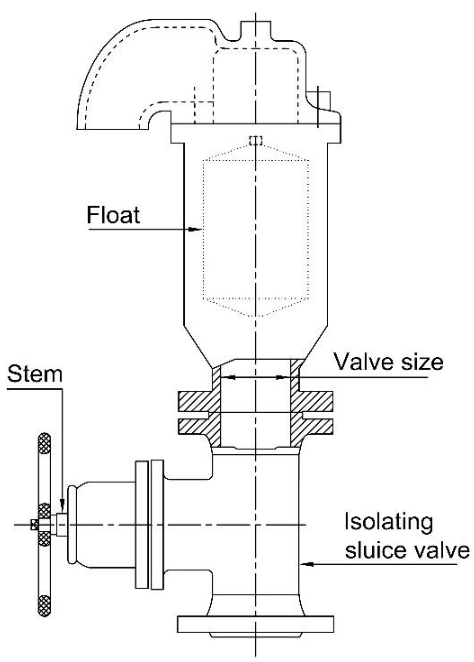
Figure 11.84: Double Air Valve with screwed down isolating valve



Major component	Material of construction
L.P.& H.P. chamber/L.P.& H.P.cover	Cast iron/Ductile iron
L.P. & H.P. ball	Vulconite/Rubber Covered timber ball
L.P./Seat	Elastomer
H.P. orifice	Gun metal
Isolating sluice valve	DF metal seated/Resilient seated
Isolating sluice valve stem	Stainless steel: AISI 410/AISI 304/AISI 316

Valve size	50Ø	80Ø	100Ø	150Ø	200Ø

Figure 11.85: Single Chamber Triple Function Air Valve



Major component	Material of construction
Body/Top cover	Ductile iron
Float assembly	S.S, AISI, 304
Seal	Elastomer
Isolating sluice valve	DIDF with resilient wedge
Isolating sluice valve stem	Stainless steel: AISI 420/AISI 410/AISI 304/AISI 316

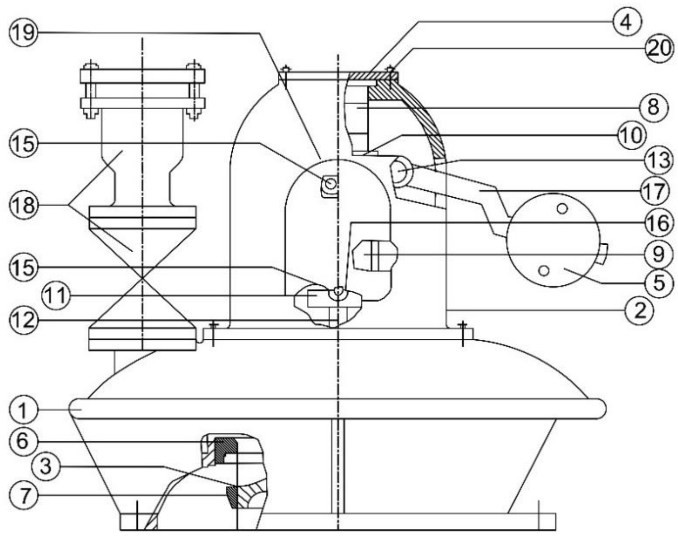
Valve Size	40Ø	50Ø	80Ø	100Ø	150Ø	200Ø

Figure 11.86: Combination of Air Valve (Tamper Proof)

(E) Anti-Vacuum Valve

The anti-vacuum valve is a very special type of air valve. Its primary function is to prevent the formation of vacuum in hydroelectric penstocks or large diameter water mains, which might cause line collapse under such conditions of flow as may result, for example, from too rapid a closure of an upstream head gate or shut down valve, a downstream burst, turbine "runaway" or ordinary emptying of a pipeline.

By virtue of its unique design, the anti-vacuum valve reacts automatically, sensitively, and positively, even after long period of inactivity, to changes of pressure within a pipe, and whenever necessary, permits air to flow in at a sufficiently high velocity, and at a low enough induction pressure, to safeguard the line against collapse. Figure 11.87 shows anti-vacuum valve.



Sr.no	Component	Qty.	Material
1	Body	1	C.I. ASTM A-126 Class B
2	Dome	1	C.I. ASTM A-126 Class B
3	Valve	1	C.I. ASTM A-126 Class B
4	Top cover	1	C.I. ASTM A-126 Class B
5	Weight	4	C.I. ASTM A-126 Class B
6	Body seat	1	Bronze BS:1400 Gr. LG2
7	Valve seat	1	Bronze BS:1400 Gr. LG2
8	Dash pot (top)	1	Bronze BS:1400 Gr. LG2
9	Dash pot (bottom)	2	Bronze BS:1400 Gr. LG2
10	Dash pot cover	1	Bronze BS:1400 Gr. LG2
11	Dash pot base plate	1	Bronze BS:1400 Gr. LG2
12	Stem	1	S.S. ASTM A-276 Type 316
13	HInge pin	1	S.S. ASTM A-276 Type 316
14	Guide pin(not shown)	1	S.S. ASTM A-276 Type 316
15	Set pin	4	S.S. ASTM A-276 Type 316
16	Lever	2	M.S.
17	Bell crank	2	D.I. ASTM A-536
18	BDO K.A.V. with I.S.V.	1	Std.
19	Inspection cover	2	C.I. ASTM A-126 Class B
20	Fasteners		S.S. AISI 304

Figure 11.87: Anti-Vacuum Valve

(F) Cowled Inlet Type

An annular cowl shrouds the orifice, affording protection to the orifice and the seating, air flows through the ports provided around the periphery of the body assembly. Application of this type should be confined to situations where no damage is likely to occur to surrounding structures from sudden intake of air.

(G) Duties

The valve automatically allows induction of large volume of air to prevent vacuum formation and also provides an automatic means of ventilating a line when it is being emptied of water and of exhausting air when it is being recharged.

(H) Operation

The valve element is in the form of a disc which is sensitively balanced by a counterpoising mechanism. The disc guide pin is attached to a crosshead, to which is fitted at either end a cranked lever that rocks about an intermediate pivot pin and carries an adjustable counterweight on its outer arm. The parts are so arranged that by adjusting the position of the counterweights, the valve can be balanced at any desired point in its working travel. Thus, when swinging freely, the valve may be balanced at partially open position in which case, if it is closed by hand, it is self-opening to the predetermined point of equilibrium and vice versa. Also attached to the crosshead is an oil dashpot which gives free opening, in a downward direction, but offers resistance to closing, in an upward direction and avoids all possibilities of oscillation of the suspended masses. In action, therefore, the valve cannot remain at either extremity of its travel unless it is acted upon by some external force. During normal operation, the disc is held shut by the water pressure in the pipe. Should the pressure on the underside of the disc fall below that of the atmosphere, the valve will immediately open to admit air and break the vacuum. With a very small vacuum, say 1 inch of mercury or about ½ psi below atmosphere, the valve opens fully and offers a wide passage for the free flow of air. On the cessation of air inflow, the valve returns to a position of slightly open, which is sufficient for the escape of air during refilling of the line. When the rising water makes contact with the underside of disc, closure is completed; only a very small water pressure is required to close the valve. Consequently, the quantity of water overflowing through the orifice during final closure is negligible.

(I) Locating Air Valve on Pipeline

The presence of free air in pipeline can reduce the severity of water hammer considerably. Celerity (speed) of an elastic wave with, say 2% of air at a pressure head of 50 m of water reduces celerity from about 1100 m/s to 160 m/s for a typical pipeline. On the other hand, air in a rising main, whether in solution or in bubble form, can have a number of ill effects.

Cavitation: The formation of vacuous cavities which subsequently rapidly collapse and erode the pump or pipe

Head loss: Accelerating water past air pockets formed in pipes, leading to head losses.

Other forms may include surging, corrosion, reduced pump efficiency, malfunctioning of valves or vibrations.

A pipeline designer therefore has to factor in all the aforesaid, the critical level of permissible vacuum inside a pipeline to prevent collapse I the suction and discharge capacity of an air valve (curves on request) before deciding upon the type, size, spacing, and location of these equipment. Figure 11.88 shows various locations where air valve may be provided in the alignment.

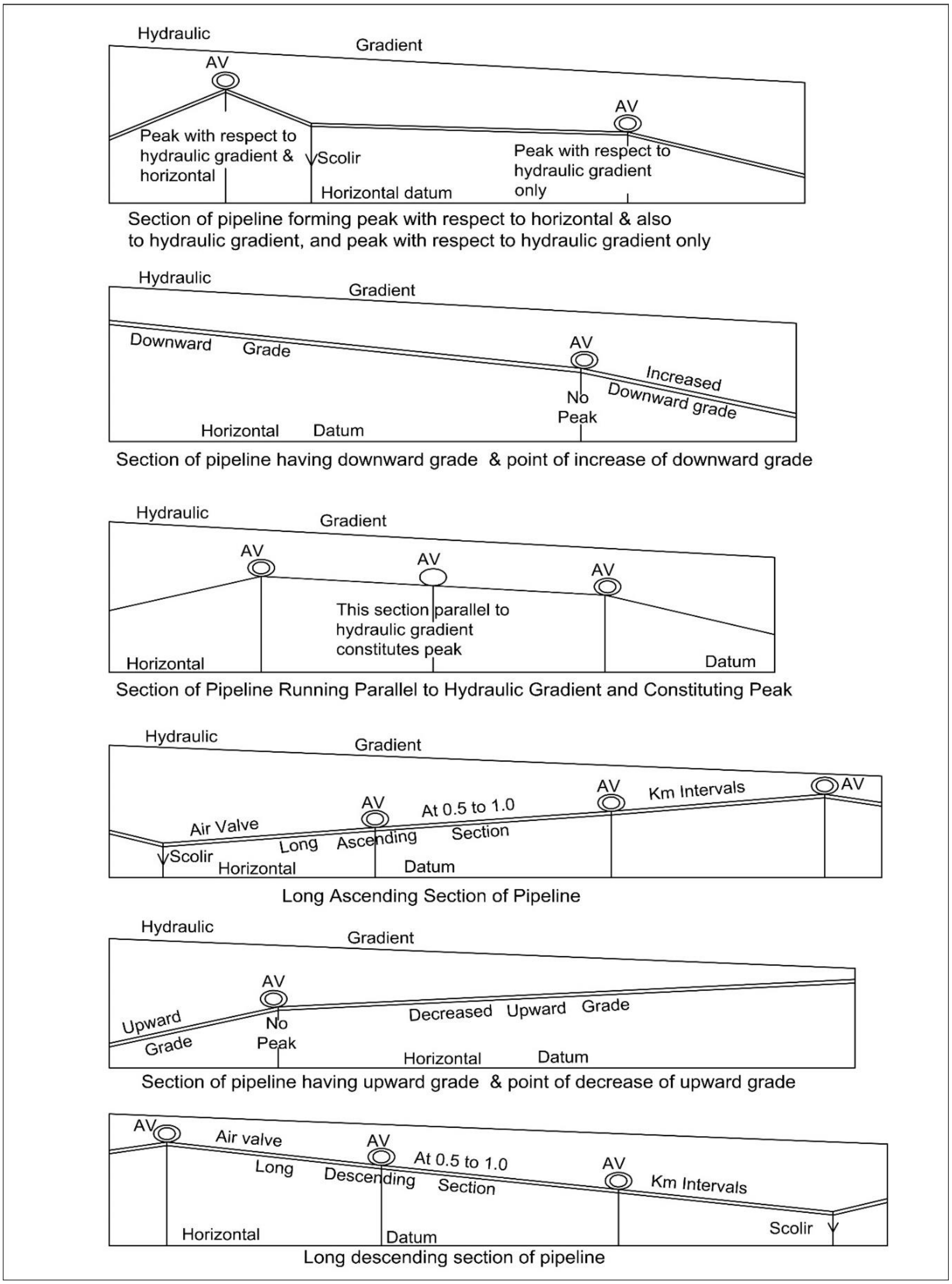


Figure 11.88: Various Locations where Air Valve may be provided in the alignment

11.23.1.7 Pressure Relief Valves

These, also called as overflow towers, are provided in one or more summits of the conveyance main to keep the pressure in the line below given value by causing water to flow to waste when the pressure builds up beyond the design value. Usually, they are spring, or weight loaded and are not sufficiently responsive to rapid fluctuations of pressure to be used as surge protection devices. The conventional pressure relief valve is characterised by a rapid pop action or by opening proportionally to the increase in pressure with respect to the opening pressure of the valve.

The main parts of the conventional pressure relief valve include the body, bonnet, disc, disc holder, seat, and spring. Based on the seating material, conventional pressure relief valves are classified as metal-seated valves and soft seated valves. See the image for clarity.

There are three types of pressure relief devices:

1. Reclosing-type pressure relief devices
2. Non-reclosing type pressure relief devices
3. Vacuum relief devices

(A) Safety Relief Valves

- A safety relief valve has a combined characteristic of a safety valve and a relief valve. It performs as a safety valve, open by pop-up action when used in a compressible gas system and performs like a relief valve, opens in proportion to the overpressure when used in liquid systems.
- Different types of safety relief valves are used in process piping.
- These valves are classified as conventional type, pilot-operated, balanced bellows type, power actuated, and temperature actuated type. Figure 11.89 shows pressure relief valve diagram.

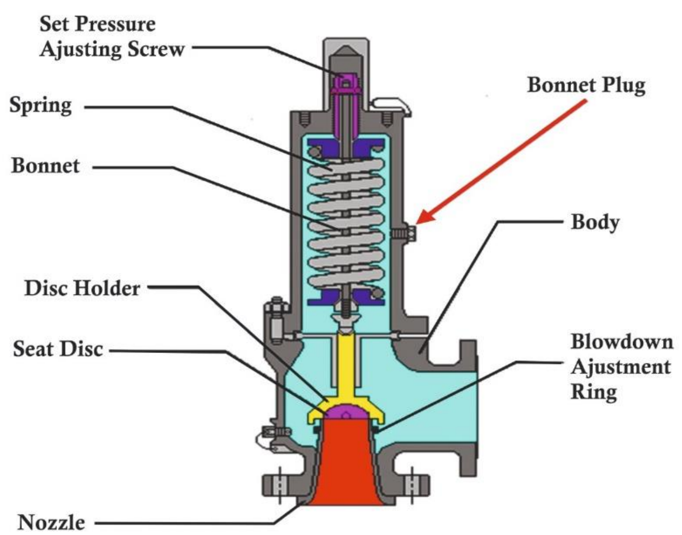


Figure 11.89: Pressure Relief Valve Diagram

During normal operating conditions, the pressure at the inlet is below the set pressure, and the disc remains seated on the nozzle, preventing flow through the nozzle.

The working principle of a conventional spring-loaded pressure relief valve is based on the balance of force. That means the spring load will keep the disc on the seat till the system pressure is less than the spring force.

This pressure is known as set pressure. The disc remains seated on the seat in the closed position till the inlet pressure exceeds set pressure and overcome the spring force. When the inlet pressure is reduced to a level below the set pressure spring force, close the valve.

Conventional pressure relief valves are used for applications where an excessive variable or built-up back pressure is not present. Back pressure will directly affect the valve performance. A pressure built-up on the outlet side of PSV is known as back pressure. You will learn more about back pressure in the lecture on bellows type PSV.

(B) Advantages and Disadvantage of Conventional PRV

Advantage	Disadvantage
It can be used in all kinds of gas and liquid services	Backpressure can affect the functioning of the valve
Suitable for high pressure and temperature services	Spring is subjected to corrosion if service material is corrosive

(C) Vacuum Relief Valve

A simple vent can provide protection against vacuum. Our home water storage tanks are fitted with this kind of simple vent. But in the industrial tank which stores various chemical and hydrocarbon, this simple vent may release vapour of these products in an atmosphere, which can be odorous, toxic, and potentially hazardous. To avoid such release, special vacuum relief valves are used.

(D) Pressure Vacuum Relief Valve -PVRV

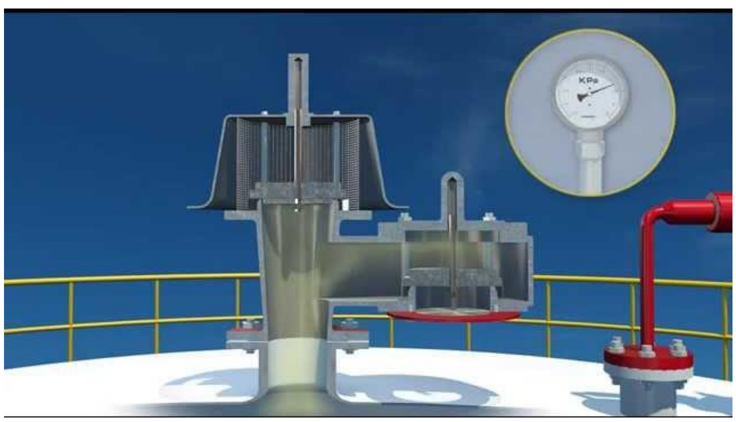


Figure 11.90: Pressure Vacuum Relief Valve -PVRV

The pressure vacuum relief valve or pressure vacuum vent are designed to maintain a tight seal until system pressure or vacuum exceeds the set pressure of the valve. When overpressure occurs, the pressure lifts the disc just like a safety valve, allowing vapours to pass. Figure 11.90 shows a pressure vacuum relief valve (PVRV).

When a vacuum occurs, vacuum lifts the disc and lets the air inside to break the vacuum. It is like breathing of a tank. This PVRV arrangement prevents continuous vapour loss to the atmosphere as it opens only when the set pressure is reached. You can see the image for a better understanding.

11.23.1.8 Diaphragm Valve

(A) The History of the Diaphragm Valve

The diaphragm valve traces its origins back to the ancient Roman and Greek times, where it was used to control the water and temperature of the hot baths. With a crude leather diaphragm that was manually closed over a weir, it was a primitive but effective control valve.

In the early 1900s, a South African mining engineer by the name of P. K. Saunders was charged with the project to cut the costly power losses due to faulty, leaking seats and stuffing boxes of the valves used to supply air and water in the underground mines. Saunders was interested in ancient history and archaeology as a hobby and stumbled upon the use of the control valves used in the baths. He utilised this concept to develop the first modern diaphragm valve. Many patents were filed in his name for this valve and in 1931, the Hills McCanna Company became the first licensee to manufacture the Saunders patent diaphragm valve in the United States. Soon after that, others entered the business such as Grinell (ITT Diameter Flow), Dow Chemical, and Arco Winn.

With the advent of a variety of advanced plastics and elastomeric materials that could be used in the internal construction of this valve, its sales growth was remarkable; however, it soon became apparent that a reliable actuator to automate it efficiently was urgently required.

Diaphragm valves (or membrane valves) consist of a valve body with two or more ports, an elastomeric diaphragm“, and a "weir ”r saddle" or seat upon which the diaphragm closes the valve. The valve body may be constructed from plastic, metal, wood or other materials depending on the intended use. Figure 11.91 shows diaphragm valve – rubber lined.

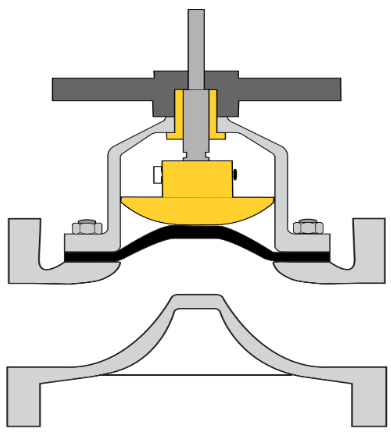


Figure 11.91: Diaphragm Valve – Rubber lined

(B) Categories

There are two main categories of diaphragm valves:

1. Weir-type (saddle) seals over a weir; and
2. Full bore or straight-through valve seals over a seat.

In general, straight-through diaphragm valves are used in on-off applications and weir-type diaphragm valves are used for control or throttling applications.

While diaphragm valves usually come in two-port forms (2/2-way diaphragm valve), they can also come with three ports (3/2-way diaphragm valves also called T-valves) and more (so-called block-valves). When more than three ports are included, they generally require more than one diaphragm seat; however, special dual actuators can handle more ports with one membrane.

Diaphragm valves can be manual or automated. Automated diaphragm valves may use pneumatic, hydraulic, or electric actuators along with accessories such as solenoid valves, limit switches and positioners.

In addition to the well-known, two-way shut-off or throttling diaphragm valve, other types include: three-way zero dead leg valve, sterile access port, block and bleed, valbow and tank bottom valve.

(C) Valve body

Many diaphragm valve body dimensions follow the Manufacturers Standardisation Society MSS SP-88. However, most non-diaphragm valves used in industrial applications are built to the ANSI/ASME B16.10 Standard. The different standards make it difficult to use diaphragm valves as an alternative to most other industrial valves. Some manufacturers offer diaphragm valves that conform to ANSI B16.10 standards thereby making these diaphragm valves interchangeable with most solid wedge, double-disc, and resilient wedge gate valves as well as short pattern plug and ball valves.

(D) Actuators

Diaphragm valves can be controlled by various types of actuators, e.g., manual, pneumatic, hydraulic, electric, etc. The most common diaphragm valves use pneumatic actuators; in this type of valve, air pressure is applied through a pilot valve into the actuator which in turn raises the diaphragm and opens the valve. This type of valve is one of the more common valves used in operations where valve speed is a necessity.

Hydraulic diaphragm valves also exist for higher pressure and lower speed operations. Many diaphragm valves are also controlled manually.

(E) Body Materials

a. Wood
b. Brass
c. Steel type:
    i. Cast Iron
    ii. Ductile iron
    iii. Carbon steel
    iv. Stainless steel
    v. Alloy 20
d. Plastic type:
    i. ABS (Acrylonitrile butadiene styrene)
    ii. PVC-U (Polyvinyl chloride, un-plasticised) also known as PVCu or uPVC
    iii. PVC-C (Polyvinyl chloride, post chlorinated) also known as PVCc or cPVC
    iv. PP (Polypropylene)
    v. PE (Polyethylene) also known as LDPE, MDPE and HDPE (see note)
    vi. PVDF (Polyvinylidene fluoride)
    vii. PTFE
    viii. PFA

(F) Body lining materials

Depending on temperature, pressure, and chemical resistance, one of the following is used:

- Unlined type
- Rubber lined type:
    - NR/Hard Rubber/Ebonite,
    - BR/Soft rubber
    - EPDM
    - BUNA-N
    - Neoprene
- Fluorine plastic lined type

  - FEP
  - PFA
  - PO
  - PP
  - Tefzel
  - KYNAR
  - XYLON
  - HALAR
- Glass lined (green glass or blue glass)

(G) Diaphragm materials

- Unlined or rubber lined type:
  - NR/Natural rubber
  - NBR/Nitrile/Buna-N
  - EPDM
  - FKM/Viton
  - BUNA-N
  - SI/Silicone rubber
  - Leather
  - Fluorine Plastic Type:
  - FEP, with EPDM backing
  - PTFE, with EPDM backing
  - PFA, with EPDM backing

(H) Applications

Diaphragm valves are ideally suited for:

- Corrosive applications, where the body and diaphragm materials can be chosen for chemical compatibility (e.g., acids, bases, etc.)
- Abrasive applications, where the body lining can be designed to withstand abrasion and the diaphragm can be easily replaced once worn out
- Solids entrained liquids, since the diaphragm can seal around any entrained solids and provide positive seal
- Slurries, since the diaphragm can seal around entrained solids and provide positive seal
- Water and wastewater
- Power
- Pulp and Paper
- Chemical
- Cement
- Mining and m
- Pharmaceutical and bioprocessing

11.23.1.9 Scour Valve (Drain Valve)

Scour valves shall be installed at low points or to facilitate draining of a water main where required by the water supply operations manager. In areas where the scouring of mains is needed as a frequent operation, a connection to the storm water curb outlets, open channels or catch it shall be provided. The connection of a scour valve to storm water pipes or manholes is not permitted. Figure 11.92 shows scour valve (drain valve).

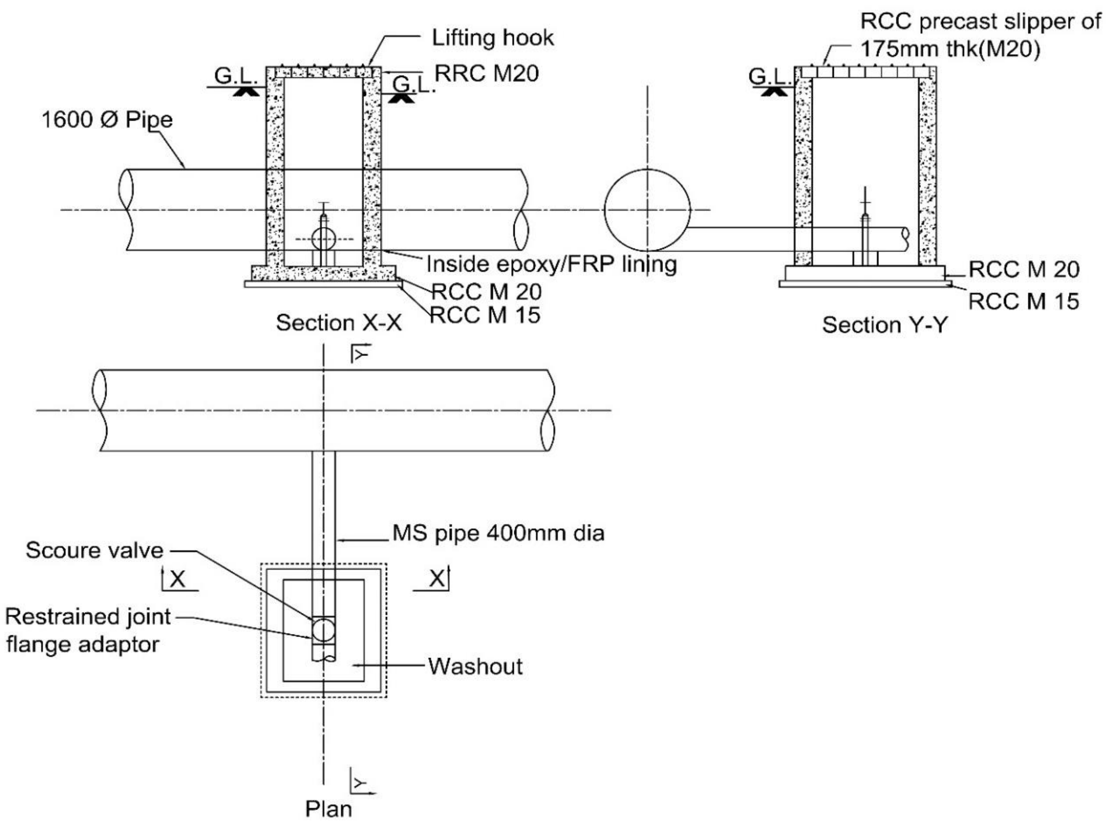


Figure 11.92: Scour Valve (Drain Valve)

11.23.1.10 Check Valves

Check valves, also called non-return valves or reflux valves, automatically prevent reversal of flow in a pipeline. They are particularly useful in pumping mains when positioned near pumping stations to prevent backflow when pumps shut down. The closure of the valve should be such that it will not set up excessive shock conditions within the system. For more details of swing check reflux valves, reference may be made to IS 5312 – Pt I - 1984 and Pt II - 1986. The valve that used to prevent backflow in a piping system is known as a check–valve. It is also known as a non-return valve or NRV. The pressure of the fluid passing through a pipeline opens the valve, while any reversal of flow will close the valve.

It allows full unobstructed flow and automatically shuts as pressure decreases. The exact operation will vary depending on the mechanism of the valve.

11.23.1.11 Pump bypass reflux valve

One of the simplest arrangements for protecting a pumping main against water hammer is a reflux valve installed in parallel with the pump. The reflux or non-return valve would discharge only in the same direction as the pumps.

Under normal pumping conditions, the pumping head would be higher than the suction head and the pressure difference would maintain the reflux valve in a closed position. On stopping the pumps, the head in the delivery pipe would tend to drop below the suction head, in which case water would be drawn through the bypass valve. The pressure would therefore only drop to the suction pressure less any friction loss in the bypass. The return wave over pressure would be reduced correspondingly.

This method of water hammer protection cannot be used in all cases, as the delivery pressure will often never drop below the suction pressure. In other cases, there may still be an appreciable water hammer overpressure (equal in value to the initial drop in pressure).

This method is used only when the pumping head is considerably less. In addition, the initial drop in pressure along the entire pipeline length should be tolerable. The suction reservoir level should also be relatively high or there may still be column separation in the delivery line. Figure 11.93 shows pump with bypass reflux valve.

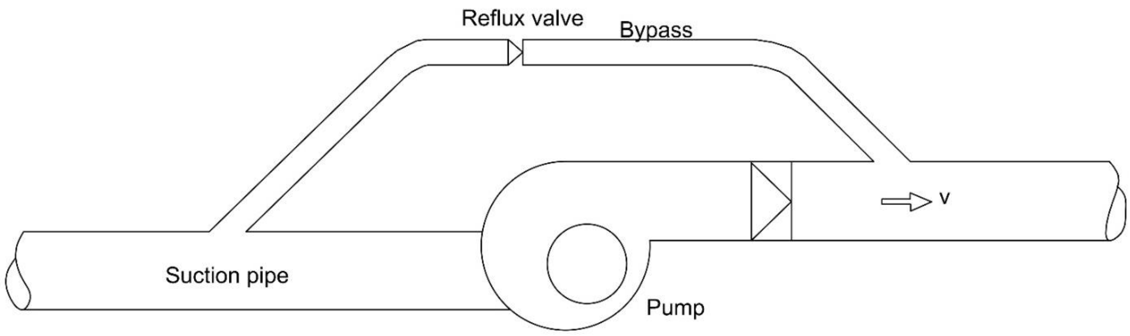


Figure 11.93: Pump with Bypass Reflux Valve

(A) Parts of non-return valve

It consists of a body, cover, disc, hinge pin, and seat ring. In the image below, you can see the parts of the valve. Figure 11.94 shows parts of NRV.

Sr No	Check Valve
1	Body
2	Cover plate
3	Disc
4	Body seats
5	Hinge
6	Hinge pin
7	Nut
8	Cotter pin
9	nuts
10	Stud bolts
11	gasket
12	Washer

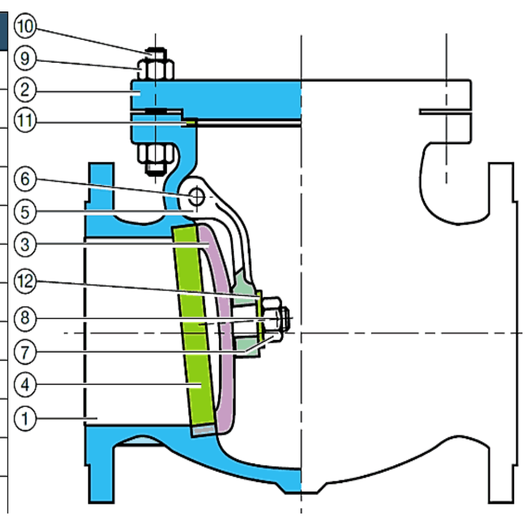


Figure 11.94: Parts of NRV

Source –f Image - TROUVAY & CAUVIN

(B) Types of check valves (non-return valve)

The type of disc will decide the type of valve. Most common types of check valves are

- Swing Type
  - Top hinged
  - Tilting disc
- Lift type
  - Piston type

- Ball-type
- Dual plate type
- Stop check valve

(C) Swing check valve

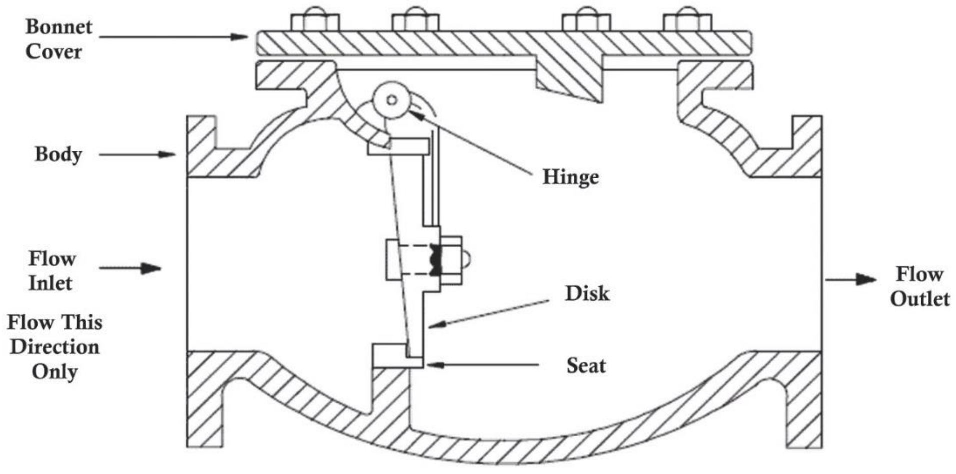


Figure 11.95: Swing Check Valve
Source of Image - DOE Handbook

The disc in a swing type valve is unguided as it fully opens or closes. This valve operates when there is flow in the line and get fully closed when there is no flow. Turbulence and pressure drop in the valve is very low. Disc and seat designs can be of metal-to-metal or metal-to-composite. Figure 11.95 shows swing check valve.

The angle between the seat and the vertical plane is known as the seating angle and varies from 0 to 45 degrees. Usually, the seat angles are in the range of 5 to 7 degrees. Larger seat angles reduce the disc travel, resulting in quick closing, thus minimising the possibility of water hammer. A vertical seat has a 0-degree angle.

The swing type valve allows full, unobstructed flow and automatically closes as pressure decreases, usually installed in combination with gate valves because they provide relatively free flow combinations.

A basic swing type valve consists of a valve body, a bonnet, and a disc that is connected to a hinge.

(D) Tilting disc check valve

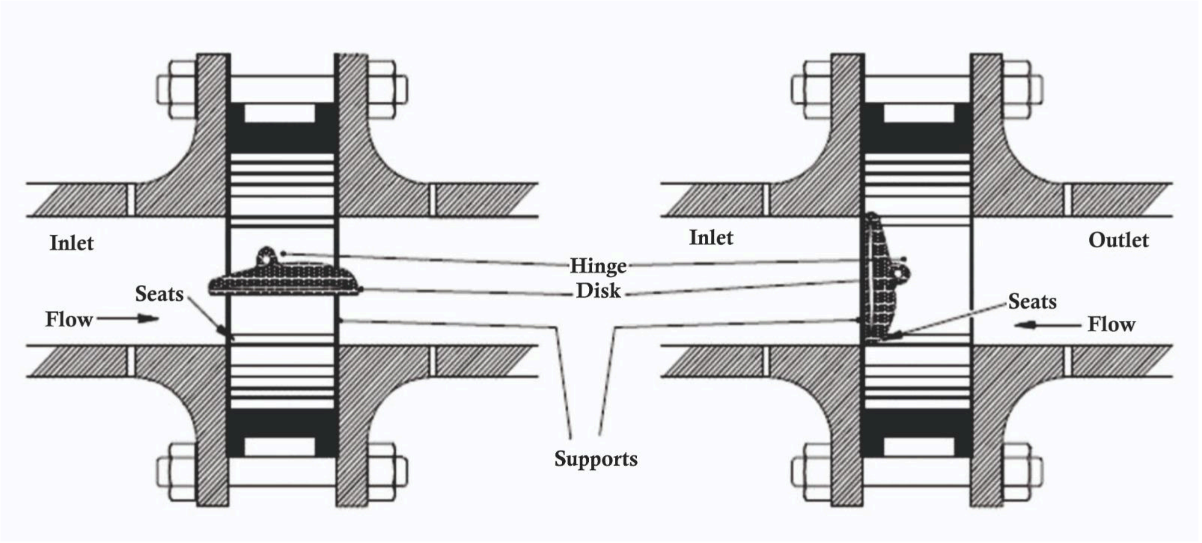


Figure 11.96: Tilting Disc Check Valve
Source of Image – DOE Handbook

The tilting disc type valve is designed to overcome some of the weaknesses of conventional swing type valves. The design of the tilting disc enables the valve to open fully and remain steady at lower flow rates and close quickly when the forwarding flow stop. Figure 11.96 shows tilting disc check valve.

The dome-shaped disc floats in the flow and fluid flow on both the bottom and top of the disc surfaces. As the disc is spring loaded, when forward flow pressure reduces, the spring force helps the valve to close fast. In the image above, you can see the flow from the valve.

Tilting disc type valve is available in wafer type and lug type design.

(E) Ball Type and plug type lift NRV

The seat design of a lift check valve is similar to a globe valve. A piston or a ball is usually used as a disc.

Lift check valves are particularly suitable for high-pressure service where the velocity of flow is high. The disc is perfectly set on the seat with full contact. They are suitable for installation in horizontal or vertical pipelines with upward flow.

When the flow enters below the seat, a disc is raised from the seat by the pressure of the upward flow. When the flow stops or reverses, the backflow and gravity forced the disc downward to set on the seat. Commonly used in piping systems that used globe valves as a flow control valve. Figure 11.97 shows plug type check valve and Figure 11.98 shows ball-type check valve.

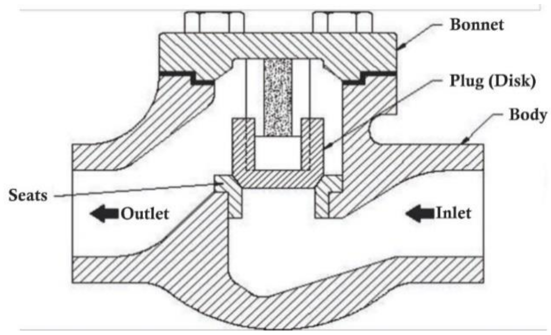


Figure 11.97: Plug Type Check Valve

Figure 11.98: Ball-Type Check Valve

Here you can see the plug or piston type and ball-type check valve. These valves provide superior leak-tight characteristics to those of swing check valves.

Some design in plug type uses spring to retain the disc in a closed position. This will ensure that the valve allows fluid flow only when there is enough pressure in the flow direction.

A ball-type valve is very simple as it simply works on the principle of gravity. When there is enough pressure in the flow, it lifts the ball upward but when pressure is reduced, the ball rolls down and closes the opening.

(F) Stop check valve

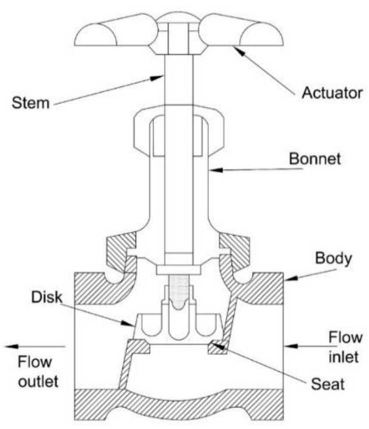


Figure 11.99: Stop Check Valve
Source of Image- DOE Handbook

Stop check valve is a combination of a lift check valve and a globe valve. It can either be used as a check valve or as an isolation (stop) valve like a globe valve. These valves can be closed with the help of a stem that is not connected to the valve disc during normal operation and make it possible to use these valves as a regular NRV. Figure 11.99 shows stop check valve.

However, when needed, the stem is used to hold the free-floating disc against the valve seat, just as a globe valve. These valves are available in tee, wye, and angle patterns. Swing type and piston lift type valves are commonly used as stop check valves.

(G) Dual plate check valves

Dual plate check valves employ two spring-loaded plates hinged on a central hinge pin. When the flow decreases, the plates close by torsion spring action without requiring reverse flow. As compared to conventional swing check valve which operates on mass movement, the Dual plate check valve are provided with accurately designed and tested torsion springs to suit the varying flow conditions. The dual plate check valves are of non-slamming type and arrest the tendency of reversal of flow. Presently, there is no IS for the dual plate check valves. Figure 11.100 shows dual plate check valve.

Figure 11.100: Dual Plate check valve

A dual plate check valve is known as a butterfly check valve, folding disc check valve, double-disc, or split disc check valve. As the name suggests, two halves of the disc move towards centreline with the forward flow and with reverse flow, two halves open and rest on the seat to close the flow (flapping action).

(H) Application of check valve (NRV)

Check valves (non-return valve) are used in a piping system to prevent backflow. The discharge line of rotary equipment such as pump and compressor always fitted with a check valve to prevent backflow. Use of dual plate check valve is popular in low-pressure liquid and gaseous services. Its light weight and compact construction make it a preferable choice when space and convenience are important. It is 80% to 90% lighter than the conventional full body check valve. It is frequently used in systems that used butterfly valves. The cost of installation and maintenance is very low compared to other types.

Properties of a dual plate check valve:

a. Compact, light, self-supporting, rugged design, and economical.
b. Low-pressure drops, thus, lower energy loss.
c. Total flexibility in installation – horizontal to vertical.
d. Valve closer at zero velocity – hence no valve induced water hammer.
e. Inherently non-slamming design without any external fitments.
f. Technically superior to swing check valve including multi door swing check valve.
g. Streamlined flow path.

Property of a swing check valve

a. Bulky and voluminous thus cumbersome handling and heavier supporting system.
b. Large and difficult to analyse from stress concentration point in critical application due to intricate body shape.
c. Suitable primarily for horizontal application.
d. Not possible significant pressure loss and energy loss, which is still higher for higher pressure ratings.
e. Swing restricted flow-path.
f. Always require reverse flow for closure and back pressure for effective sealing.
g. External attachments required to counteract slamming.

h. Water hammer tendency persists.
i. Seat and hinge pin require regular maintenance due to impact loads and wear by rubbing.

11.23.1.12 Ball Valves or Ball Float Valves

Ball valves or ball float valves are used to maintain a constant level in a service reservoir or elevated tank or standpipe. The equilibrium type of valve is the most effective and it is designed to ensure that the forces on each side of the piston are nearly balanced. For severe operating conditions, a more expensive needle-type valve will give better service.

In both cases, the float follows the water level in the reservoir and permits the valve to admit additional water on a falling level and less water on a rising level and to close entirely when the overflow level is reached. The disadvantage of this system is that the valve may operate for long periods in a throttled condition, but this can be avoided by arranging for the float to function in a small auxiliary cylinder or a tank. When the water reaches the top of the auxiliary tank, the ball will rise fairly quickly from the fully open position to the closed position without shock. The valve will not open again until the water level in the reservoir reaches the base of the auxiliary tank, at which point the water will drain away and the ball valve will move to the fully open position. With this method the valve is not in a state of almost continuous movement and throttling and erosion of the seats are avoided.

Automatic Shut-Off Valves

These are used on the mains to close automatically when the velocity in the mains exceeds a predetermined valve in case of accident to the line.

Automatic Burst Control

With large steel mains suitably protected against corrosion and laid properly, particularly at change of direction and the ground is not liable to subsidence, the possibility of a major burst is ruled out.

The simplest arrangement as explained in 11.25.1.7(c) is to insert an interrupter timer in the motor circuit so arranged that the final quarter travel of a sluice valve occurs in slow steps to the point of closure. The costlier arrangement will be insertion of a smaller power operated bypass valve alongside the main valve and provision of automatic control arrangements for the main valve to close first at a fairly rapid rate, followed by the smaller bypass valve at a much lower speed.

Plug Valve

Plug valve is a quarter turn rotary motion Valve that uses a tapered or cylindrical plug to stop or start the flow. The disc is in plug shape, which has a passage to pass the flow.

In the open position, this bored passage is in line with the flow. When the plug is turned 90° from the open position, the solid part of the plug blocks the flow.

It is used in place of a gate valve where the quick operation is required. It can be used in high-pressure temperature services.

(A) Types of Plug Valves

These valves are available in either a lubricated or non-lubricated design and with different styles of port openings through the plug.

a) Lubricated plug valve

The plug inside a lubricated plug valve has a cavity in the middle along its axis. You can see this in the image. Lubricant chamber at the bottom and the sealant injection fitting at the top ensure the supply of lubricant.

The small check valve below the injection fitting prevents the sealant from flowing in the reverse direction once the sealant is injected into the cavity.

Plug surface gets constantly lubricated by the sealant that moves from the centre cavity through radial holes into lubricant grooves on the plug surface.

The narrow gap around the plug may allow leakage, and if the gap is reduced further, it will increase the friction and the plug may get stuck inside the valve body. Figure 11.101 shows plug valve.

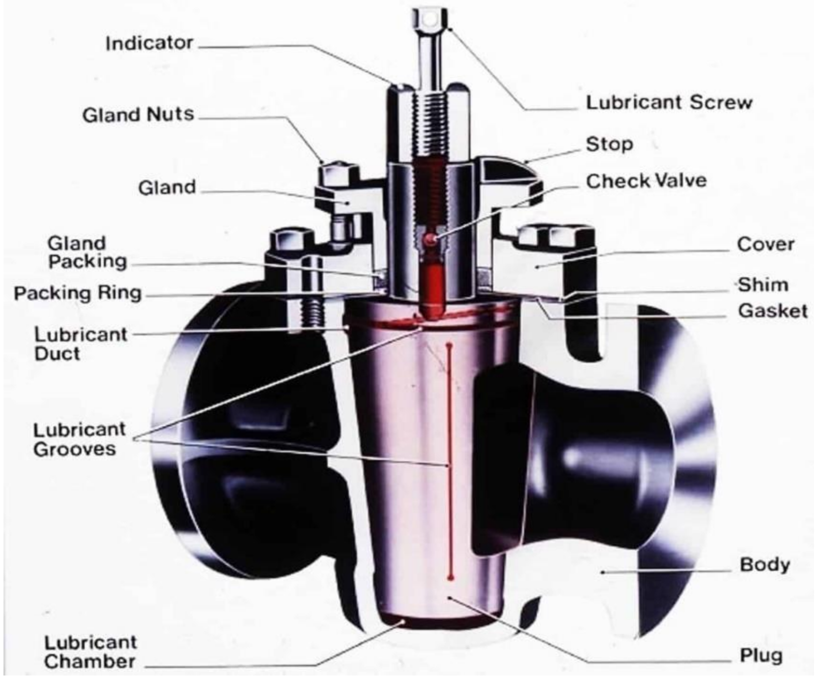


Figure 11.101: Plug Valve
Source of Image - Serck Audco, Newport

The lubricant reduces the force required to open or close the valve and allows smooth movement of the plug. It also prevents corrosion of the plug.

The lubricant material must be compatible with the fluid of the pipeline. It should not dissolve or wash away by the flow medium as this could contaminate the fluid and damage the seal between the plug and the body, resulting in leakage. Also, the sealant used must be able to withstand the temperature of the flow medium.

Lubricated plug valves are available in the large size range, and they are fit to work in high-pressure temperature services. These valves are subject to less wear and provide better corrosion resistance in some service environments.

b) Non-lubricated plug valves

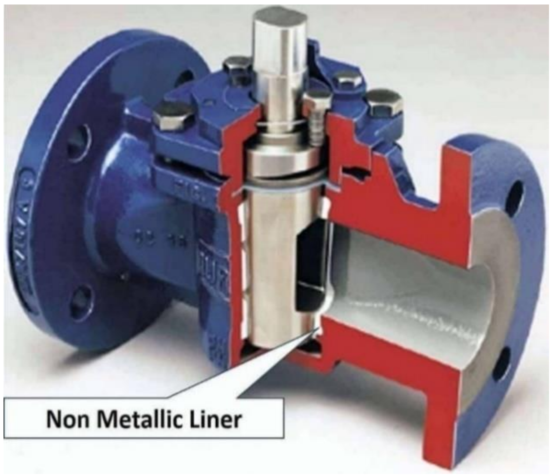


Figure 11.102: Typical non-lubricated plug valve

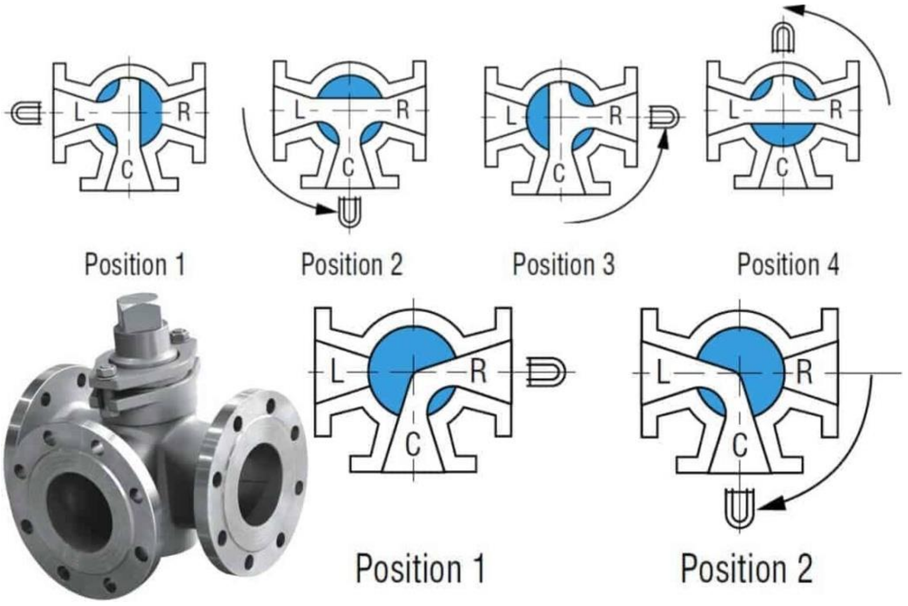
A non-metallic elastomeric sleeve or liner is used in this type of valve. This sleeve is installed in the body cavity of the valve. The polished tapered plug acts as a wedge and presses the sleeve against the body. Figure 11.102 shows typical non-lubricated plug valve.

This non-metallic sleeve reduces the friction between the plug and the valve body. Non-lubricating plug valves required minimum maintenance. Due to the non-metallic seat, these valves are not used in high-temperature services.

Lubricating and non-lubricating valves are capable of providing a bubble-tight shutoff and are of compact size.

c) Multi-port plug valves

Below is the figure of the three-way multi-port plug valve. The top image is of three-way three-port design and the bottom is a three-way two-port design. Figure 11.103 shows four-way design of multi-port plug valves.



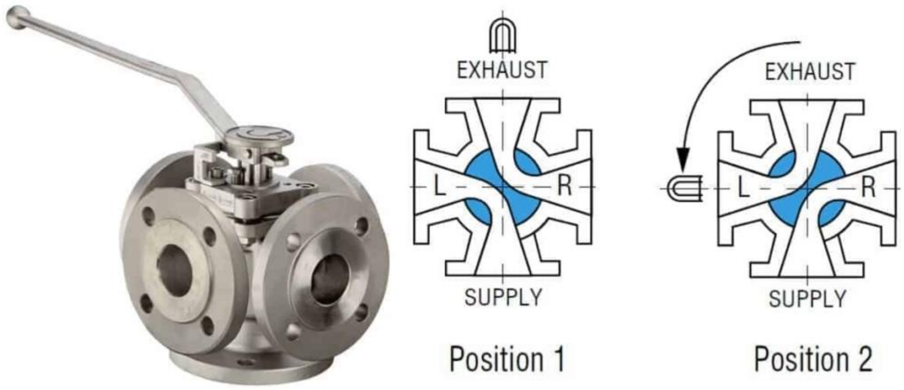


Figure 11.103: Four-way design of Multi-Port Plug Valves

Multi-port valves are used in transfer lines and for diverting services. A single multi-port valve may serve the purpose of three or four gate valves or other types of the shutoff valve.

However, sometimes the multi-port valve does not completely shut off flow. Great care should be taken in specifying the particular port arrangement for proper operation.

(B) Plug Valve Parts

The typical plug valve consists of a body, bonnet, stem, and plug. The seat is an integral part of the body in the case of a lubricated type. For a non-lubricated type, a non-metallic seat is used to improve leak tightness of the valve. Figure 11.104 shows plug valve parts.

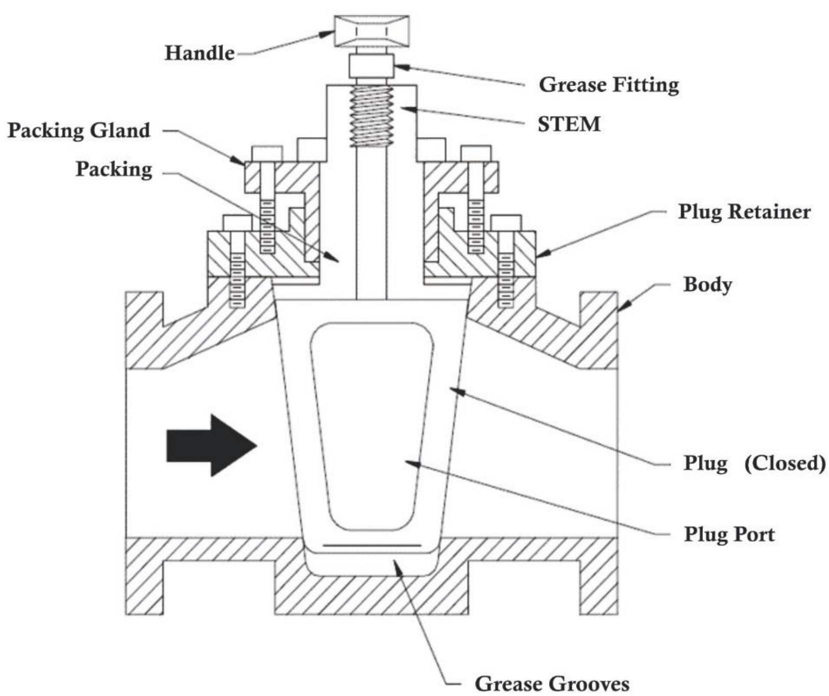


Figure 11.104: Plug Valve Parts

Source of Image – DOE Handbook Plug Valve Disc Types

Plugs are either round or taper cylinder. They may have various types of port openings, each with a varying degree of the opening area.

Plugs are available with

- Rectangular Port

- Round Port and
- Diamond Port

Rectangular Port is the most common for a plug valve. The rectangular port represents at least 70% of the corresponding pipe's cross-sectional area.

Round port plug has a round opening through the plug. It is available in full bore and reduced bore design. Valves with reduced ports are used only where restriction of flow is not important.

Diamond Port plug has a diamond-shaped port through the plug. All diamond port valves are venturi restricted flow type. This design is for throttling service.

(C) Application of valve

- This valve is used as on-off stop valves and capable of providing bubble-tight shut-off.
- It can be used in different types of fluid services such as air, gaseous, vapour, hydrocarbon, slurries, mud, and sewage applications.
- Also used in a vacuum to high pressure and temperature applications.

(D) Advantages and disadvantage

Advantages	Disadvantages
Simple design with few parts	It requires greater force to operate, due to high friction
Quick to open or close	Larger valves cannot be operated manually and required an actuator
In-line maintenance possible	Pressure drop due to reducing port
Offers minimal resistance to flow	Cost of plug valves may be more than ball valves for given size and class
Provides reliable leak-tight service	
Multiple port design helps reduce the number of valves needed and permits a change in a flow direction	

11.23.1.13 Smart Valves

(A) General

The Hydraulic Engineers were using sluice valves, gate valves, or butterfly valves from years together, not only for isolation but also for regulating the flow of water on ground. It was a regular practice to throttle the valves by adjusting the threads to control the flow. This adjustment was completely dependent on the years of experience of the valve person. Even though regulation of flow to some extent could be achieved, still, the lacuna of accuracy existed.

With the advent of modern technology, the manual adjustment by means of throttling was replaced by electrical actuators. Although human intervention got reduced, but the operation cost due to power consumption increased. Also, the valves were unable to function in case of power failure.

To isolate and drain pipe sections for testing, installation, cleaning and repairs, a number of appurtenances or auxiliaries are generally installed in the line like line valves, scour valves, air valves, kinetic air valves, pressure relief valves, check valves, etc.

To overcome all the difficulties faced due to the aforesaid valves, concept of control valves was introduced. Precision in controlling flow, pressure and other hydraulic parameters could now be achieved. Human intervention has been reduced and the hydraulic engineer got an option to select between hydraulically and electrically operated valves or electronic valves depending on the geographic conditions of the service area in the cities/towns.

Control valves are smart valves used to control the hydraulic parameter (flow/pressure) at varying degrees between minimal flow and full capacity in response to command from a control device. The hydraulic parameter is controlled by varying the size of flow passage as regulated by the control device. This also enables consequential steering of combination of hydraulic parameters.

Problems are inevitably faced by field engineers related to management of water in highly elevated areas, distant service reservoirs, untimely filling of reservoirs, uncontrolled water supply (pressure, flow, and level management in distribution network) and reduction in non-revenue water (NRW). These aforesaid problems result in unequal distribution of water in the service areas. If control valves used smartly, it shall resolve most of the hydraulic issues as mentioned above. Sizing of these types of valves should be decided on the basis of required hydraulic applications.

In order to procure the optimal size and to ensure no cavitation, the following information must be provided for each main valve size required in the project:

a) Upstream pressure available,
b) Maximum flow rate,
c) Minimum flow rate,
d) Residual pressure at the control valve,
e) Maximum level of reservoir (in case of elevated service reservoir management).

(B) Functions of Control Valves and its Types

Depending on the purpose of its service, control valves may be classified as under:

1. **Flow control valve (FCV):** The valve sets a fixed flow at the downstream. An adjustable differential pilot valve, sensing the pressures across the orifice plate, is set to maintain a preset differential, hence the flow rate. The pilot opens the main valve if the flow is below the required value or closes it when the flow exceeds the desired rate. Figure 11.105 shows typical FCV.

Figure 11.105: Typical FCV

2. **Pressure reducing valve (PRV):** The valve sets a fixed pressure at the downstream when opened. An adjustable pilot valve, sensing the pressures downstream of the valve, is set to maintain a preset pressure regardless of fluctuations upstream. (Note: The calculated cavitation factor of a valve should be higher than that claimed by the manufacturer. Else, anti-cavitation kit or step reduction of pressure by installing multiple pressure reducing valve should be deployed). Figure 11.106 shows a typical PRV.

Figure 11.106: Typical PRV

3. **Pressure reducing valve with flow control:** The valve sets a combination of fixed pressure at the downstream when opened or the fixed flow rate at the downstream when opened regardless of fluctuations in the values of upstream pressure and demand. Figure 11.107 shows a PRV with flow control.

Figure 11.107: PRV with Flow Control

4. **Level control valve (float type or altitude type):** The altitude pilot is located at the valve and float pilot is located in the tank at the intended maximal level. It is connected to the main valve by control tubes, which convey the inlet pressure of the valve to the control chamber when the level reaches the maximal point, thus, closing the valve and preventing overflow. When the water level drops, due to demand from the tank, the altitude/float pilot opens the top of the control tube that connects to the control chamber, allowing the water in it to drain and the main valve to open. Figure 11.108 shows a level control valve.

[Cavitation factor sc = $(P_1 – P_v)/(P_1-P_2)$ where P_1 = upstream pressure, P_2 = downstream pressure, P_v = Vapour Pressure]

Figure 11.108: Level Control Valve

5. **Flow control valve with timer:** This is a flow control valve with timer controller to set the times of operation without any external energy. The valve opens and shut as per the programmed

time. The valve can be actuated with a 9V battery as well if a latching solenoid (NC) is placed before the rate of flow control pilot. Figure 11.109 shows an FCV with timer.

Figure 11.109: FCV with timer

6. **Flow control valve with level control:** This is a flow control valve with capability to control level. When the level is high, valve closes and when level reaches the set point, it opens. The valve sets a fixed flow at the downstream when opened, the valve can be installed at the ERS/OHT inlet as well. The valve will maintain the tank level and prevent overflowing as well as feed the tank at a preset and a relatively a fixed flow rate. The valve can help in equitable filling up of the water towers connected to common feeders. Figure 11.110 shows an FCV with level control.

Figure 11.110: FCV with level control

7. **Pressure sustaining and relief valve:** The valve maintains upstream pressure, regardless of flow-rate variations. The valve will be in the “closed” position if the upstream pressure drops below the set point and will fully open when the upstream pressure exceeds the set point. The valve helps in sustaining the HGL as well as pressure to a critical user. Figure 11.111 shows a pressure sustaining and relief valve.

Figure 11.111: Pressure Sustaining and Relief Valve

8. **Surge anticipation valve:** The valve is installed on a tee junction on the discharge manifold or the main pipeline, downstream of the check valves of pumping station. The valve opens instantly when the pressure at site drops below the static value due to initial low-pressure wave generated by the pump stoppage. The valve stays in “opened” position until the returning flow arrives to the station and will be sized to allow draining of part of it. The velocity changes of the returning flow, when stopped by the already closed check valves, will not generate excessive transient pressures. When the pressure rises above the opening point, the valve starts closing gradually at adjustable pace. In case of too-fast closure, that may cause pressure surge, the valve will function as a relief valve, re-opening slightly to increase the released flow rate thus preventing overpressure in the pump site.

   The above listed functions are controlled by hydromechanical pilot valves – low pressure pilot that opens the valve through the low-pressure wave, and high-pressure pilot which opens the valve in case the pressure exceeds the allowed maximum. Both pilots are adjustable by the operator. A simulating assembly, containing a pressure gauge, should be a part of the control circuit. It allows adjustment of the low-pressure pilot to the designed opening point, without stopping the pumps.

   A drainpipe should be assembled at the outlet of the surge anticipation valve to transport the drained water back to the pumps suction pit or to another location, where the high velocity flow will not cause flood damage. To cease high hydraulic resistance and excessive side-stress in elbows, the drainage pipe should be sized to avoid excessive velocity. In most of the cases, the drainage pipes are larger than the control valves. The valve can be electrically timed as well where the static head is low. In such case, the low-pressure sensing pilot is replaced with a solenoid and a special surge anticipating panel and a UPS for energising the solenoid valve. Figure 11.112 shows surge anticipating valve with solenoid and Figure 11.113 shows dual pilot surge anticipating valve.

Figure 11.112: Surge Anticipating Valve with Solenoid

Figure 11.113: Dual Pilot Surge Anticipating Valve

9. **Pump control valve:** A pump control valve automatically regulates the pump start-up and shutdown in a time controlled to minimise system hydraulic surges. The pump control valve is electrically interfaced with the pump motor, the pump control valve “opens” and “closes” at an adjustable speed, providing a smooth, predictable transition of pump discharge flow into the system. The valve is installed at the outlet of pump discharge. The valve is controlled by an electric solenoid valve. The pump control valve automatically adjusts to provide constant pressure at different flows and works with any pump. (Note: However, the utilisation of these valves is neither mandatory nor limited to the applications cited. The engineer can use the control valves as per his discretion after verification of the hydraulic conditions of the site). Pump control valve needs to be operated by a special panel for opening, closing sequencing co-ordinated with the main pump panel. Figure 11.114 shows a pump control valve.

Figure 11.114: Pump Control Valve

(C) Design

All the above mentioned control valves from a) to h) are available in two types of design which are the diaphragm actuated control valve and the plunger type control valve. These are explained in following sections.

(D) Diaphragm Actuated Control Valve

This type of valve can further be classified into following categories:

I) Globe-type control valve

All the functions, viz., opening, closing, and regulation is done by stem assembly actuated by diaphragm. It contains manyparts like stem, seat, diaphragm, centring guide, etc. When the ratio of upstream and downstream pressure is more than 3:1, globe- type valve can be used with anti-cavitation kit. This is a robust valve and can handle high-pressure differentials in the water network. Such valve can also be used as check valve. Although the diaphragm in this type of valve requires replacement once in seven years. The dynamic O-ring can get worn off if there is problem in water quality and thereby leading to internal leakage. Since it involves so many parts, this valve is comparatively heavy.

II) Weir-type control valve

All the functions, viz., opening, closing, and regulation is performed by the diaphragm. The diaphragm and the spring are the only moving parts, thereby making it lighter and easier to repair. The diaphragm needs replacement once in five years. When the ratio of upstream and downstream pressure is more than 3:1, weir-type valve can be used in a step-down manner implying utilisation of two or more valves and thereby leading to an increase in capital cost.

Both the globe-type and weir-type valves are again available in the following three types such as:

(a) **Hydraulically pilot-operated control valve –** This is a pilot-operated valve. The number of pilots is dependent on the application, such that the control parameters are not hydraulically contradictory. This pilot will be a direct acting adjustable spring device, maintaining any set point as the prescribed value, regardless of flow or upstream pressure variations. If thevalve has to perform the function of both level and flow control, there would be two pilots – one for the flow and the other for the level. The pilot governing the closure would be considered as master pilot. For example, in case of pressure reducing cum flow control valve, if the fixed and set pressure of 45 m attains prior to attaining the set flow rate of 50 cum/hr., the pilot designated for pressure would dictate the closure of the valve and would be called the master pilot. This type of valve does not require electricity and should be preferred where the set points are fixed.

(b) **Hydraulically pilot-operated control valve with single solenoid.** This is a hydraulically controlled valve by SCADA or local PLC with the provision of an additional solenoid in the control loop to enable On/Off functionality. It is suitable for situations where it is necessary to control valve operations through the SCADA system with fixed set-point control.

(c) **Electronic (dual solenoid) control valve** – There is no pilot required inthis type of valve. This is a dual solenoid operated valve, and the performance is governed by a controlling device that can be PLC, RTU, or preferably, inbuilt controller. The controller should be able to receive either a pulse/volume contact-input or a 4-20mA analogue signal. The flow/pressure set value may be modified automatically on time basis, or by a predefined relation to pressure or another measured parameter. This should also be operated by smart android phone or local HMI screen for change of parameters and to avoid compatibility with the systems. (Note: The performance of the electronic (dual solenoid) control valve for flow controlapplication depends on the performance of flowmeter. Hence, selection of flowmeter should be done as per flowmeter manufacturers' guidelines.)

Technical comparison of all aforesaid three types of valves has been represented below in Table 11.5.

Table 11.5: Technical comparison of all aforesaid three types of valves

S. No.	Features	Hydraulically Pilot-Operated Control Valve	Hydraulically Pilot-Operated Control valve with Single Solenoid	Electronic (dual solenoid) Control Valve
1	Material	Ductile Iron one-piececast body	Ductile Ironone-piece cast body	Ductile Iron one-piece castbody
2	Flow/Pressure control	Yes	Yes	Yes
3	Level Control	Yes	Yes	Yes
4	Manual on/off	Yes	Yes	Yes

S. No.	Features	Hydraulically Pilot-Operated Control Valve	Hydraulically Pilot-Operated Control valve with Single Solenoid	Electronic (dual solenoid) Control Valve
5	Flowmeter	Not required	Not required	Yes
6	Level Transmitter	Not required	Not required	Yes
7	Power Required	No	Battery operated	Solar/Grid energy
8	SCADA operation	Not possible	Yes, for on/off purpose	Yes
9	O&M Cost	Minimal	Only to change AAA batteries	Maintenance of solar panels, battery replacement
10	Requirement of external devices	None	None	Flowmeter/level transmitter/pressure transmitter to control flow/level and pressure respectively
11	Failure	Low	Low	Failure in case of no power supply or failure of flowmeter or level transmitter
12	Set Points	Fixed set point	Fixed set point	Variable set points
13	Controller/RTU/PLC and Communication	Not required	Required	Required
14	Suitability	Urban, Rural.	Urban, Rural.	More suitable to urban given the power dependency and more sophisticated instruments involved
15	Manpower	Not required	Not required	Not required
16	Vertical operation	Any Position	Any Position	Any position
17	Programming	NA	Minimal	Programming knowledge is required for third party controller. Not required forinbuilt controller.
18	Recurring cost	None	None	SIM card charges for communication
19	Main line strainer	Not required	Not required	Not required

Thus, both globe-type and weir- type are diaphragm- operated valves, performing the same function, differing only in their design.

11.23.1.14 Plunger Type Valve

This type of valve can also perform the functions of regulating hydraulic parameters (flow, pressure, and level) like diaphragm actuated control valve. This type of valve requires a gear box and actuators for its operation. Since it requires continuous power consumption, the same is not preferred. Although this valve can perform all functions of valves mentioned in Section 3, at present, this type of valve is not much in vogue.

The hydraulic engineers have to first define the purpose of installation of valves before its selection. After which, depending on the geographical/site situation, these may be chosen among the options available in the diaphragm and plunger type valves.

(A) Specifications

I) General requirements

The control valve shall be designed and hydro-tested for the 1.6 times the rated pressure of the control valve, i.e., PN16 rated valve should be tested for pressure of 25 kg/cm^2. The control valves shall be designed to cause minimum head loss in fully opened condition. Flange ends should be as per IS-1538/ISO16/ ANSI B-16.5, Class150 and Class 300/EN-1092-2 or any other National/International Standards. The material of all components of valve/internal working parts shall be corrosion-resistant in case of chlorinated water. The epoxy coating (both external and internal) should be fusion-bonded, food grade of minimum 250 microns (NSF / FDA / WRAS approved).

According to the water network technology's advancement and evolution, the valve may be upgraded on site, i.e., hydraulic valve to electronic (dual solenoid) control valve or flow control valve to pressure reducing valve, by just changing the control trim. The internal parts of the valve body, stem, seating material, package, disc, plug, back rings, etc., are collectively referred to as the trim. In other words, all wetted parts which can be removable and replaceable in the valve body are part of the control valve trim.

More specifically, function of the valve may evolve towards other standards of control in terms of hydraulic or electronic modulation.

A control valve must demonstrate the following main features:

- **Sensitivity** (ability to respond to the smallest change of the controlled variables);
- **Accuracy** over time within the prescribed operating range.

Stability within the prescribed operating range, for the low demand conditions, must be specified for the lower limits/worst conditions below which the valve may beunable to operate at full stability.

The manufacturer may indicate a simple rate of flow, or a formula allowing for its calculation, and accounting for the pressure differential through the valve. The accuracy of control valve is very important. The regulating valve shall warranty the highest ranges of accuracy when required and same must be checked before approving the control valves during inspection of the project, which should be indicated in absolute value, according to the following Table 11.6.

Table 11.6: Regulating Valve Accuracy Ranges

Downstream Pressure Control	Downstream pressure not more (or less) than ±1 m of water column of set value
Upstream Pressure Control	Upstream pressure not more (or less) than ±1 m of water column of set value
ON-OFF altitude level control	Maximum reservoir level not more than ±0.10 m of water column of set value

Flow control	Downstream flow not more (or less) than ±5% of defined flow rate.

The manufacturer may specify and describe:

- the recommended operating range of the regulating valve by indicating clearly its extreme limits;
- the design features allowing for a precise and stable operation of the valvewithin this range; and
- the way of setting the operating speed of the valve.

(B) Detailed specifications

I. Hydraulically operated diaphragm-type control valve with globe pattern

The detailed specification of a hydraulically operated diaphragm-type control valve with globe pattern is as under:

a) Main valve

The valve should be preferably single chamber/double chamber, straight/oblique pattern hydraulically operated, direct diaphragm actuated stem guided globe-type. The main valve consists of three major parts: body, cover, internal trim assembly. The only moving part is the internal trim, which is guided in two ends. The stem should be fully guided at both ends by a dismountable stainless steel bearing in the main valve cover. It is bottom -guided, allowing it to slide within the valve-seat in order to be in balance along its full lift for limiting excessive friction, thus increasing its ability to react when performing a correction. This is the recommended solution; however, slotted disc guide is acceptable.

The main valve should be suitable for low flow conditions and capable of operating in unsteady flow conditions. No supplementary mechanical device may be allowed for achieving stable operation at near zero flows. The diaphragm assembly should be the only moving part and should form a sealed chamber in the upper portion of the valve, separating the operating pressure from line pressure. Packing glands and/or stuffing boxes technology should not be permitted and there should be no piston operating the main valve.

b) Material of Construction for PN10, PN16 and PN25 rated Control Valves with Globe Pattern is given in Table 11.7 below:

Table 11.7: Material of Construction for PN10, PN16 and PN25 rated Control Valves with Globe Pattern

Body and cover	Ductile iron ASTM A536, ENGJS GGG50 or equivalent
Circuit fittings	Brass EN12164/SST316/SS304 (corrosion proof)
Diaphragm	Nylon reinforced rubber/EPDM/Buna-N (FDA/WRAS approved)
Control tubes	High-pressure polypropylene/Copper/SST316/SST304
Pilot and relay	Brass EN12164/SST316/SS AISI-304/CF8 or bronze
Surface protection	Epoxy coating min. 250 microns fusion-bonded, food grade (NSF/FDA/WRAS approved)
Operation	Automatic, manual override enabled with bonnet exhaust barrel to cater any emergency of water supply. By exhausting bonnet, the valve becomes fully open with no water on top of the diaphragm.
Seat ring	Stainless steel AISI – 304/316, raised (if applicable), bronze replaceable in-line or onsite, ASTM-A351 GR CF8M

Stem	Stainless steel AISI-304/316 (replaceable in-line and onsite)
Spring and bearing bush	Stainless steel AISI-302/304/316
Disc guide, disc retainer and diaphragmwasher	Cast steel, stainless steel AISI – 304/316, bronze and coated steel, DI EN GJS GGG50 or equivalent
Seal	Synthetic rubber-Buna-N /EPDM (if applicable) (FDA/ WRAS approved)
Self-cleaning filter	Stainless Steel 316/CF8/SST304/transparent heavy duty with flushing mechanism.
Solenoid valve (if applicable)	IP68 for underground condition and IP65 if vertical installationon ESR (brass, SST 316 base). IP68 solenoids must be warranted for infinite time under 1m submerged condition.
Throttling plug	To have the linear flow (non-turbulent flow), if required, V-shaped or U-shaped throttling plug may be provided
Nut-bolts and studs	Stainless steel AISI-304/ASTM A 193 B7/A2
Air vent valve with isolating valve Provision	The cover of the valve shall have the provision to fix the small air vent valve so as to release the trapper air in the diaphragm and to maintain the sensitivity and ensure required performance of the diaphragm.

Manufacturer must provide the Kv value of each diameter of the valve for selection of valves. The standard valve model fits all control operations (pressure reducing, level control, flow control, or combination of parameters). Disassembly and reassembly of all the valve's components shall be made possible on site, without having to remove the valve from the line. The valve's pilot control loop should include a low maintenance, in-line self-cleaning control filter. The valve should also be suitable for vertical assembly wherever required. Oblique/Y-shaped valves over 200 mm diameter should not be preferred due to dismantling difficulties. Double chamber valve should be selected where external pressure is required to open the valve. The valve shall include a low friction trim. No O-ring sealing is permitted on the valve stem. The valve should require low maintenance.

c) Maintenance
- The bidder should propose a recommended five year set of spare parts per a batch of five valves of the same diameter.
- The valve should be built in a way that enables all future maintenance action to be performed in situ without having to take the valve body of the line.
- The typical weight of internal assembly, regardless of valve diameter, should not exceed the permitted lifting weight for a single person as defined in the regulations.
- Disassembly should not require usage of sophisticated, heavy lifting devices such as cranes of any type. These are to be provided and installed at the assembly site by the supplier.

II. Hydraulically operated, diaphragm-type control valve with weir pattern

The detailed specification of a hydraulically operated, diaphragm-type control valve with weir pattern is as under.

a) Main valve

Valve should be a single chamber hydraulically operated weir-type control valve. Manufacturer must provide the Kv value of each diameter of the valve. The valve should consist of three major components: the body, cover, and the diaphragm assembly. The diaphragm trim with a stainless steel spring should be the only moving part. The diaphragm should form a sealed chamber in the upper portion of the valve, separating operating pressure from line pressure. The standard valve model fits all control operations (pressure reducing, level, flow control or combination of parameters). The diaphragm of the weir-type valve should not be guided by any shafts or bearings and should not be in close contact with other valve parts except for its sealing surface. A compressing spring at the top side of the diaphragm is essential to a reliable closure.

Disassembly and reassembly of all the valve's components should be made possible on site, without having to remove the valve from the line. The valve should also be suitable for vertical assembly wherever required. The main valve should be suitable for low flow conditions and capable of operating in unsteady flow conditions. No supplementary mechanical device may be allowed for achieving stable operation at near zero flows. The selection of valve may ensure a positive drip-tight shut-off. The valves must pass hydro-test or 1.6 times the rated pressure of the control valve.

b) Material of construction for PN10, PN16, and PN25 rated control valves with weir pattern is given in Table 11.8 below:

Table 11.8: Material of Construction for PN10, PN16 and PN25 Rated Control Valves with Weir Pattern

Body& cover	Ductile iron ASTM A536 or equivalent
Circuit fittings	Brass EN12164/SST316/SST304
Diaphragm	Nylon reinforced rubber/EPDM/Buna-N (FDA /WRAS approved)
Control tubes	High-pressure polypropylene/Copper/SST316/SST 304
Pilot and relay	Brass EN12164/SST316/CF8/SST304
Surface Protection	Epoxy coating min. 250 microns, colour RAL 5005 Blue
Operation	Automatic, manual override enabled
Spring and BearingBush	Stainless Steel AISI – 302/304/316 (if applicable)
Seal	Synthetic rubber-Buna-N /EPDM (if applicable)
Self-Cleaning filter	Stainless steel 316/ CF8/SST304/transparent heavy duty with flushing mechanism.
Solenoid Valve (if applicable)	IP68 solenoid for underground condition and IP65 solenoid if vertical installation on ESR (Brass, SST 316 base). IP68 solenoids must be warranted for infinite time under 1 m submerged condition.

c) Maintenance

- The bidder should propose a recommended five year set of spare parts per a batch of five valves of the same diameter.
- The valve should require low maintenance. No set periodic packing or parts replacement should be required.
- The valve's pilot control loop should include a low maintenance, in-line "self-cleaning" control filter.
- The typical weight of internal assembly, regardless of valve diameter, shall not exceed the permitted lifting weight for a single person as defined in the regulations. Disassembly should not require usage of sophisticated, heavy lifting devices such as cranes of any type. These are to be provided and installed at the assembly site

by the supplier. Figure 11.115 shows weir-type valve design and Figure 11.116 shows valve with stem and seat ring globe straight pattern.

Figure 11.115: Weir-Type Valve Design

Figure 11.116: Valve with Stem and Seat Ring Globe Straight Pattern

III. Hydraulically operated diaphragm-type control valve with single solenoid:

This is also hydraulically operated control valve with an additional solenoid for on/off purpose through SCADA or local PLC/RTU. Solenoid should be IP 68 for underground condition and IP 65 for vertical installation on ESR (brass, SST AISI 316 base). IP 68 solenoids must be warranted for 10 years under 1 m submerged condition. It is highly recommended to provide a bypass tubing for solenoid valve for any emergency manual operation. Preferably, rigid type SS 304 control tubing and fittings should be used. The valve must have provision for a small air vent valve on top of the cover to ensure the proper functioning of the diaphragm. Figure 11.117 shows hydraulically operated diaphragm-type control valve with single solenoid.

Figure 11.117: Hydraulically Operated Diaphragm-Type Control Valve with Single Solenoid

IV. Dual solenoid operated diaphragm-type control valve

There are no pilots for this type of valve. This is a dual solenoid operated valve, performance of which should be governed by a controlling device through PLC, RTU or preferably inbuilt controller. In other words, an electronic (dual solenoid) control valve should be controlled by an electronic controller, which enables control of the requested flow rate or pressure as per need and demand. This is a very smart valve and can perform multi-functions through a command from the RTU/PLC. Multiple logics can be formulated and this smart valve can manage pressure, level, leakages, and flow operations.

A controller designed for the control of hydraulic control valves with the support of two continuous operation solenoid valves and/or two latching-type solenoid valves. The configuration should enable high regulation accuracy as well as energy efficient operation. The controller must have inbuilt battery to run the controller for a minimum of one week during no-power condition. Accordingly, it should also log the data during no-power condition. The controller should enable selection among the solenoid valve types: normally open and/or normally closed. It should allow user to set the variable set points in a timetable as required during supply hours (peak, normal hours). Set values are modifiable with the help of remote communication or an analogue signal. In case of disconnected control signal, the controller would be able to continue to regulate per preset internal values. The controller must have four digital inputs, four analogue inputs and one RS-485 connection. The user should be able to allocate control inputs and outputs, as per the used control functions so that each input can be used for one or more control functions. The control function should be user-configurable and/or pre-configurable by the manufacturer/supplier. The function should include, but not be limited to, ‘Pressure Reducing’,‘Pressure Sustaining’, ‘Flow control’, ‘Water Level control’, ‘Pressure Relief’ and combination of these parameters. It is highly recommended to provision for bypass tubing for each solenoid valve for any emergency manual operation. Preferably, rigid type SS 304 control tubing and fittings should be used. The valve must have provision for a small air vent valve on top of the cover to ensure the proper functioning of the diaphragm. Figure 11.118 shows dual solenoid operated diaphragm-type control valve.

Figure 11.118: Dual Solenoid Operated Diaphragm-Type Control Valve

The change in requirement (pressure to flow control) should not require change in any hardware or software of the systems. Combination of two or more of these control function should be possible without any restriction, as long as this combination is not logically contradicting and sufficient number of inputs are free. The user may not be required to have preliminary programmingknowledge or have other special expertise. To avoid issues in compatibility, the controller and the control valve should be preferred of the same manufacturers/suppliers. The size of the controller should be compact in order to avoid operational difficulties. It should be ensured that the programming language of the RTU/PLC is easy to understand, modifiable, and should not be monopoly driven by the interest of the valve suppliers. Knowledge transfer should be seamless and non-dependent on any specific software that is controlled by the valve suppliers. The transition between the various operators shall be easy in the interest of the projects. The reputed PLCs that are easily available and sustainable should be preferred in the larger interest of the O&M of the project.

I. Electrically operated plunger type valves

a) Main valve

Plunger valve should be provided with electrical actuators having the control facility for intermediate valve positioning by connecting external signal. The electric actuators should be designed to provide the required torque for operations in the flow and pressure conditions of the water transfer system. Gear assembly should be provided as necessary. The flow path with annual flow cross-section in any open position should be rotationally symmetric. The movement of plunger/piston by means of crank/shaft/spindle drives should be axial/linear. A handle wheel should be provided for plunger valves so that operations of the valve can be carried out when the power supply of valve is failed. The torque requirements at the hand wheel should be such that one person can operate the valve. Hand wheel should be positioned to give access for operational personnel. It should be provided with integral locking device to prevent operation by unauthorised persons. A selector switch should be provided on the actuator for remote/local/hand operation of the valve. Valve/profile sealing seat should be preferably in the no-flow zone. The O-ring seal should envisage the double sealing effect between the body and the plunger. There should not be any obstruction in the main flow passage except the plunger and attached control cylinders.

b) Material of construction electrically operated plunger type valves is given in Table 11.9 below:

Table 11.9: Material of Construction Electrically Operated Plunger Type Valves

Body	Ductile iron ASTM A536/DI (GJS 500-7)
Plunger/Piston	Stainless steel AISI -304/Gr 1.4301
Piston Guide	Bronze welded overlay/SS
Shaft Crank/Spindle	Stainless Steel AISI-420/Gr 1.4021
Seat Ring	Stainless Steel AISI-316/Bronze
Seal (O-ring)	Synthetic rubber-Buna-N/EPDM (FDA/WRAS approved)
Bearing Bush	Bronze
Bolts	Stainless steel (A4)
Eye Bolt for Lifting	Galvanised steel
Slotted cylinder/Strainer	Stainless steel
Coating (both inside and outside)	Epoxy coating min. 250 microns, fusion-bonded, food grade (NSF/FDA/WRAS approved)

The hydraulic engineer, when opting for diaphragm actuated control valves, can include both the specifications of globe pattern and weir pattern for procurement purpose. The Bureau of Indian Standards (BIS) may also be followed, if available for these types of valves.

(C) Advantages

The degree of perfection, precision and accuracy which can be attained with respect to regulation by using control valves can hardly be achieved manually or by using actuators. The valves can reduce utilisation of manpower without demanding extra maintenance cost. They can also help in controlling non-revenue water (NRW). As a result, the benefits of these valves are too many and to be analysed based on cost benefit ratio. Yet one can find out the savings in workforce and other indirect expenditures as given in Table 11.10 below:

Table 11.10: Format for Finding Savings in workforce and Other Indirect Expenditures

S. No.	Particulars	Amount (Rs.)	Remarks
Total Cost incurred prior to installation of Control Valves			
1	Expenditure incurred towards establishment of employees who are needed for operation of valves		
2	Indirect Losses – Cost of NRW		Total Water Lost(MLD) × Cost of treated water Rs/MLD × 365
Total Cost:			**(A)**
Total cost to be incurred after installation of control valves			
1	Expenditure to be incurred towards establishment of employees who are needed for operation of valves		
2	Indirect Losses – Cost of NRW		NRW will not be nil
Total Cost:			**(B)**
Saving: (A) – (B)			

However, it is suggested that procurement of valves like flow control, pressure reducing, pressure sustaining, and surge anticipation shouldn't be decided on the basis of savings in cost alone. It should also be decided on cost benefit ratio and life cycle cost assessment.

(D) Limitations

Control valves are viewed as a threat by the ground level staff because of the opinion of ground staff that the workforce shall be curtailed. Control valves may also control the tampering of the valves because of its availability for different hydraulic conditions. In absence of technical knowledge of the working of the control valves, it is not utilised to the best of its potential thereby leading to unfruitful expenditure. Hence, prior to installation of valves, the staff of Urban Local Bodies should be given technical training and build the capacity to implement the same at ground level.

The hydraulic diaphragm-type control valves require minimum hydraulic pressure between 5 m and 12 m (varying from manufacturer to manufacturer) for its operation in water supply systems. It is the duty of the hydraulic/environmental engineer to ensure that there is some residual head as specified by the manufacturer for flow control and pressure reducing applications. From the maintenance point of view, it is recommended to install isolation valves as per the requirements.

(E) Applications of Control Valves

The hydraulic/environmental engineer can use the abovementioned control valves in the following scenarios:

I. Case 1: The entire demand of the city is catered through only one ESR (Please Refer Figure 11.119)
Depending on the requirement, the hydraulic/environmental engineer can effectivelyuse either one or all the valves suggested in the sketch or in combination. If the supply is through pumping, surge anticipation valve can be installed for surge protection and water hammer. The level control valve at the inlet of ESR can be used for reducing human intervention and NRW. For intermittent supply, hydraulic/environmental engineer can install a flow control valve with timer whereas the same can be replaced by an electronic (dual solenoid) control valve in case of continuous supply. The workforce saved by adopting this method can be used for other activities.

II. Case 2: There are two or more ESRs in the system (Please refer Figure 11.120).
Same as above. Depending on the difference in ground levels of the ESRs, the level control valves at the inlets of ESRs can be replaced by a flow with level control valvefor equitable distribution of water.

III. Case 3: Water is supplied to the ESRs through a master balancing reservoir (MBR) (Please refer Figure 11.121).
Same as Case 2.

IV. Case 4: The city is big and there are multiple district metering areas (DMAs) under each ESR (Please refer Figure 11.122).
Same as Case 3. For equitable distribution of water beyond ESR, flow control valve should be installed at the entrance of each DMA.

V. Case 5: The city is huge and there are multiple sub-zones in each DMA (please refer Figure 11.123)
Same as Case 4. In addition, there is flow control valve at the entrance of each sub-zone. This will exert better control, reduce human intervention considerably and also NRW.

VI. Case 6: In hilly areas (Please refer Figure 11.124)
In hilly areas, bursting of pipelines due to high pressure is very common. To counter such type of problems like the name suggests, pressure reducing valves should be installed.

VII. Case 7: When it is mandatory to maintain certain minimum pressure in the pipeline (Please refer Figure 11.125).
When it is mandatory to maintain a certain minimum pressure on the upstream sideof a particular location, pressure sustaining valve should be installed at such location.

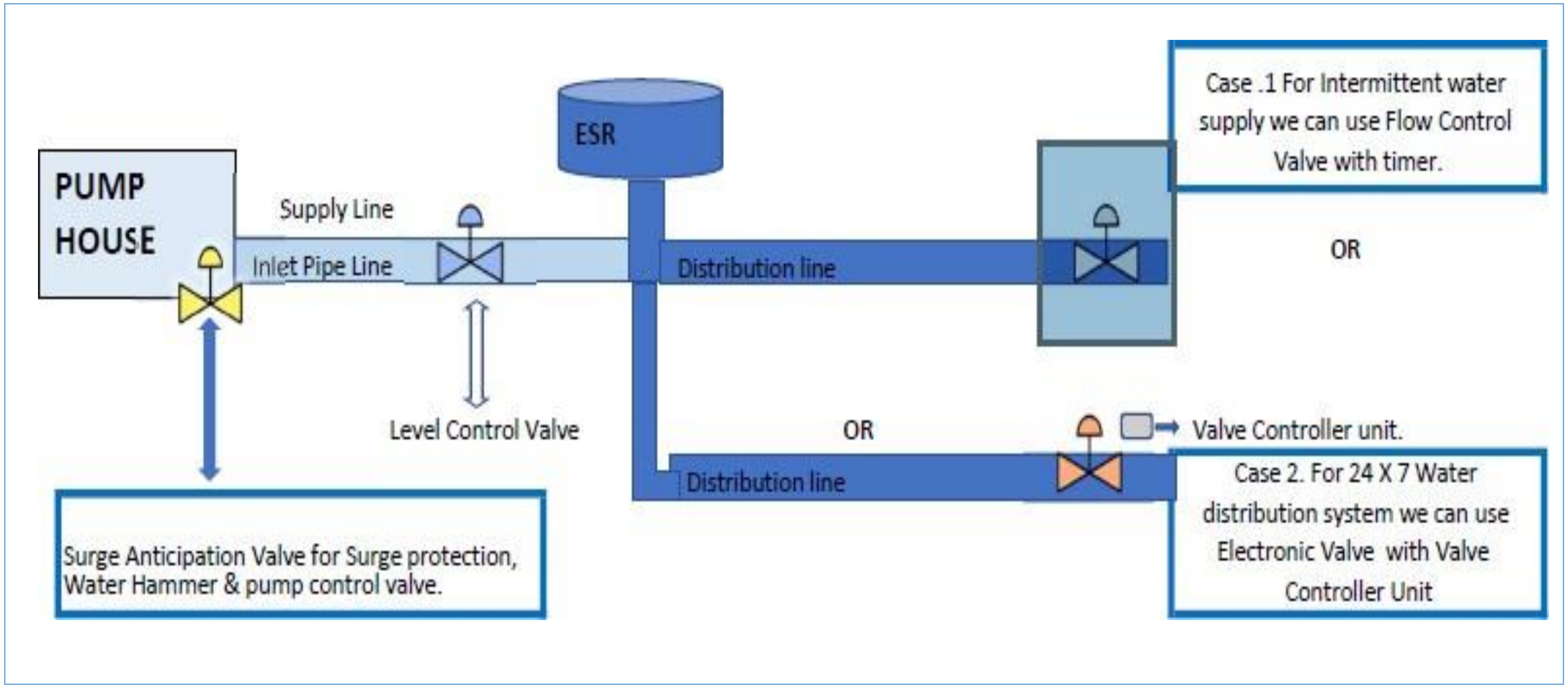


Figure 11.119: Demand of the City Catered Through Only One ESR

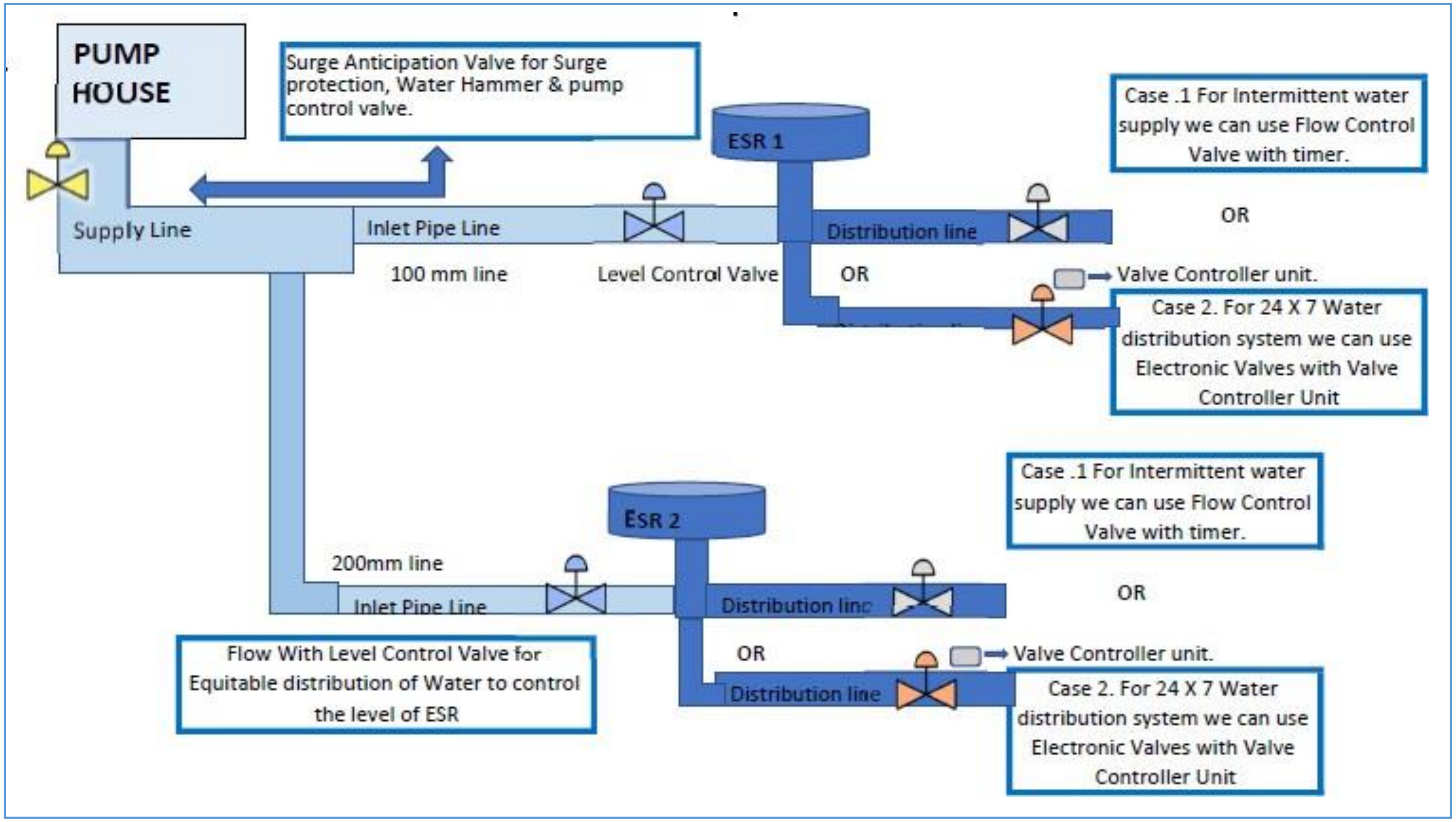


Figure 11.120: Demand of the City Catered Through Two or More ESRs

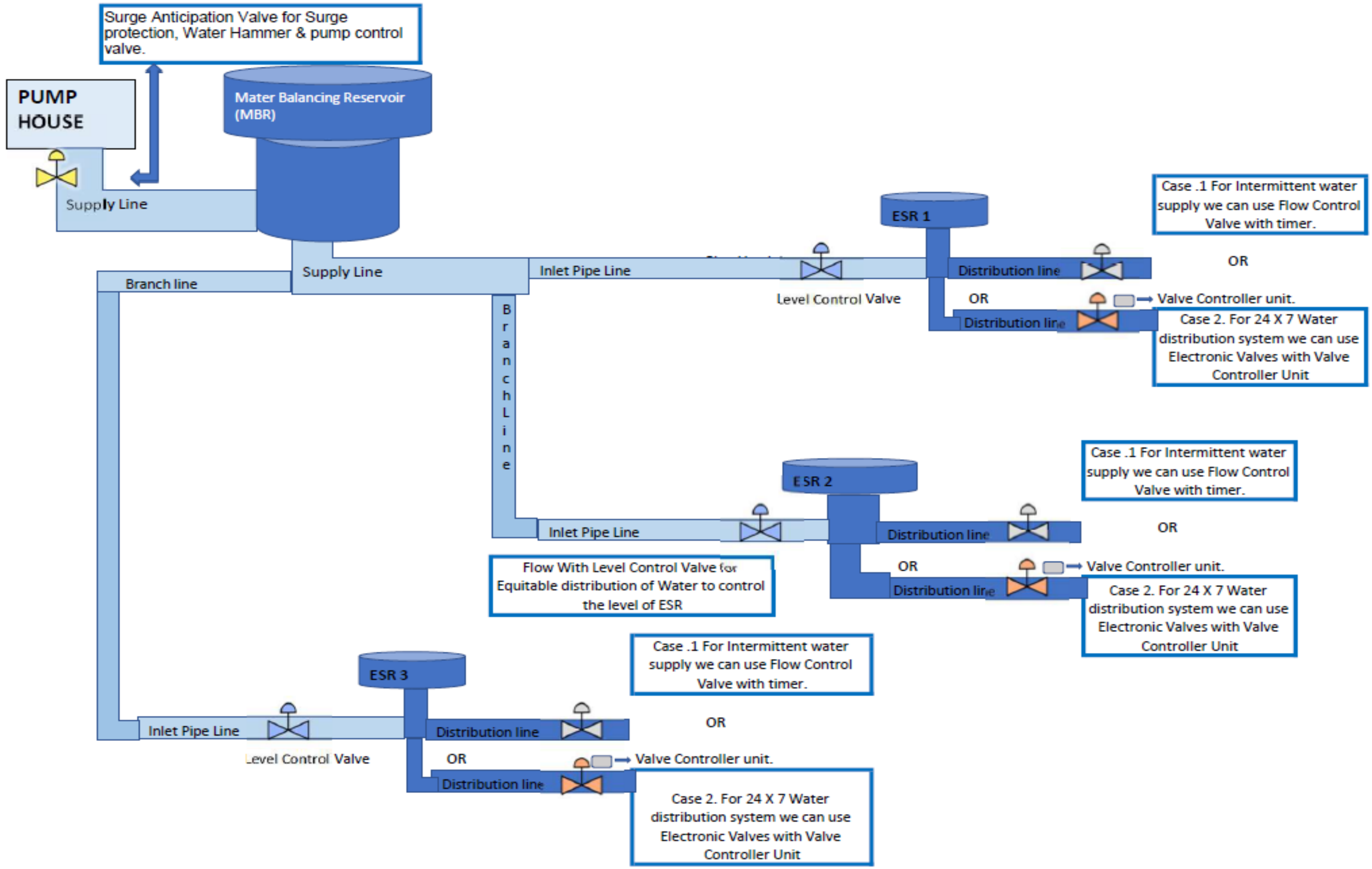

Surge Anticipation Valve for Surge protection, Water Hammer & pump control valve.
PUMP HOUSE
Mater Balancing Reservoir (MBR)
Supply Line
Supply Line
Branch line
Inlet Pipe Line
Level Control Valve
ESR 1
Distribution line
OR
Distribution line
Valve Controller unit.
Case .1 For Intermittent water supply we can use Flow Control Valve with timer.
OR
Case 2. For 24 X 7 Water distribution system we can use Electronic Valves with Valve Controller Unit
Branch Line
ESR 2
Inlet Pipe Line
Distribution line
OR
Distribution line
Valve Controller unit.
Case .1 For Intermittent water supply we can use Flow Control Valve with timer.
OR
Case 2. For 24 X 7 Water distribution system we can use Electronic Valves with Valve Controller Unit
Flow With Level Control Valve for Equitable distribution of Water to control the level of ESR
ESR 3
Inlet Pipe Line
Level Control Valve
Distribution line
OR
Distribution line
Valve Controller unit.
Case .1 For Intermittent water supply we can use Flow Control Valve with timer.
OR
Case 2. For 24 X 7 Water distribution system we can use Electronic Valves with Valve Controller Unit

Figure 11.121: When Water is supplied to the ESRs through a Master Balancing Reservoir (MBR)

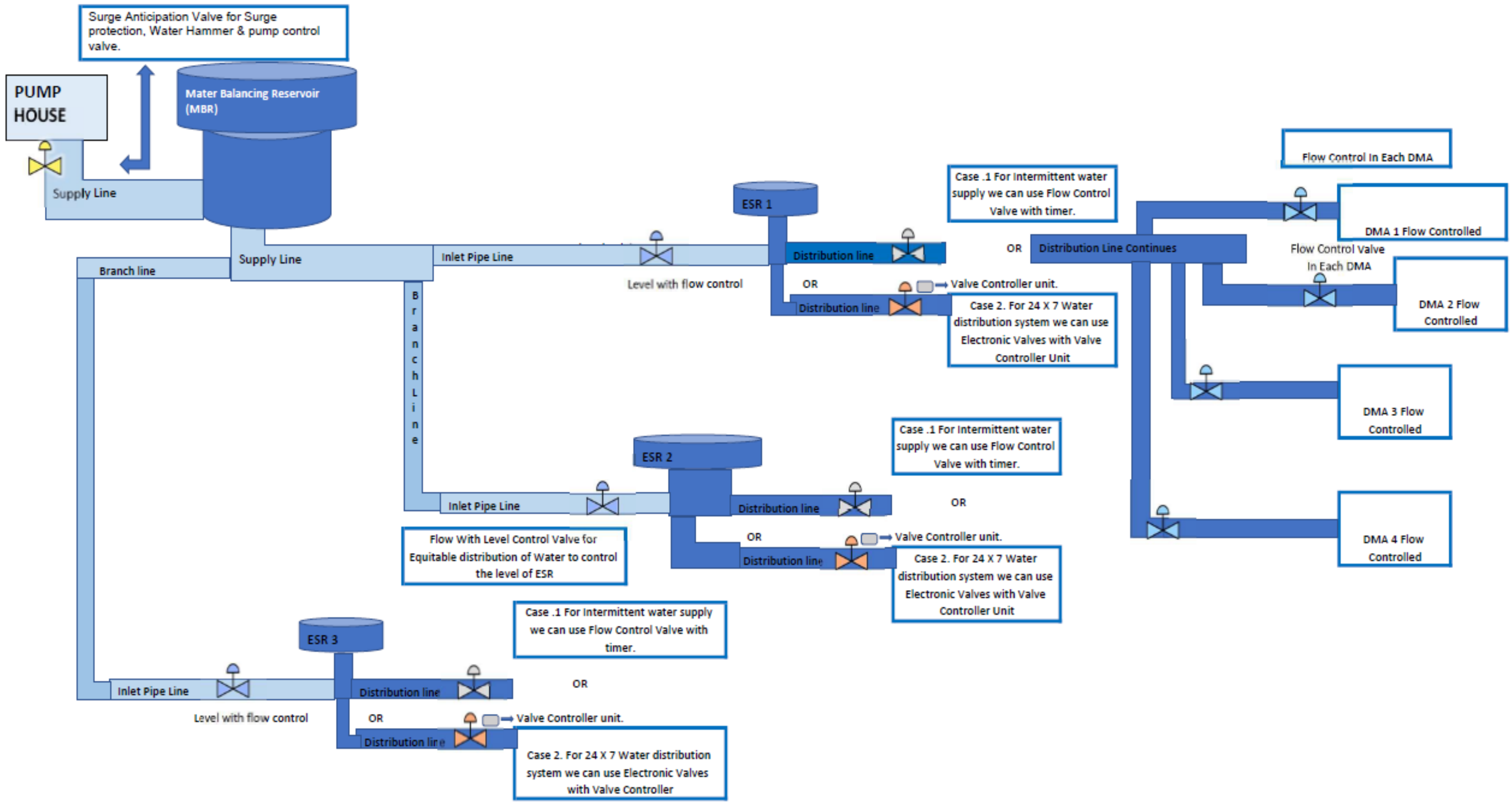


Figure 11.122: City is big and there are multiple District Metering Areas (DMAs) under each ESR

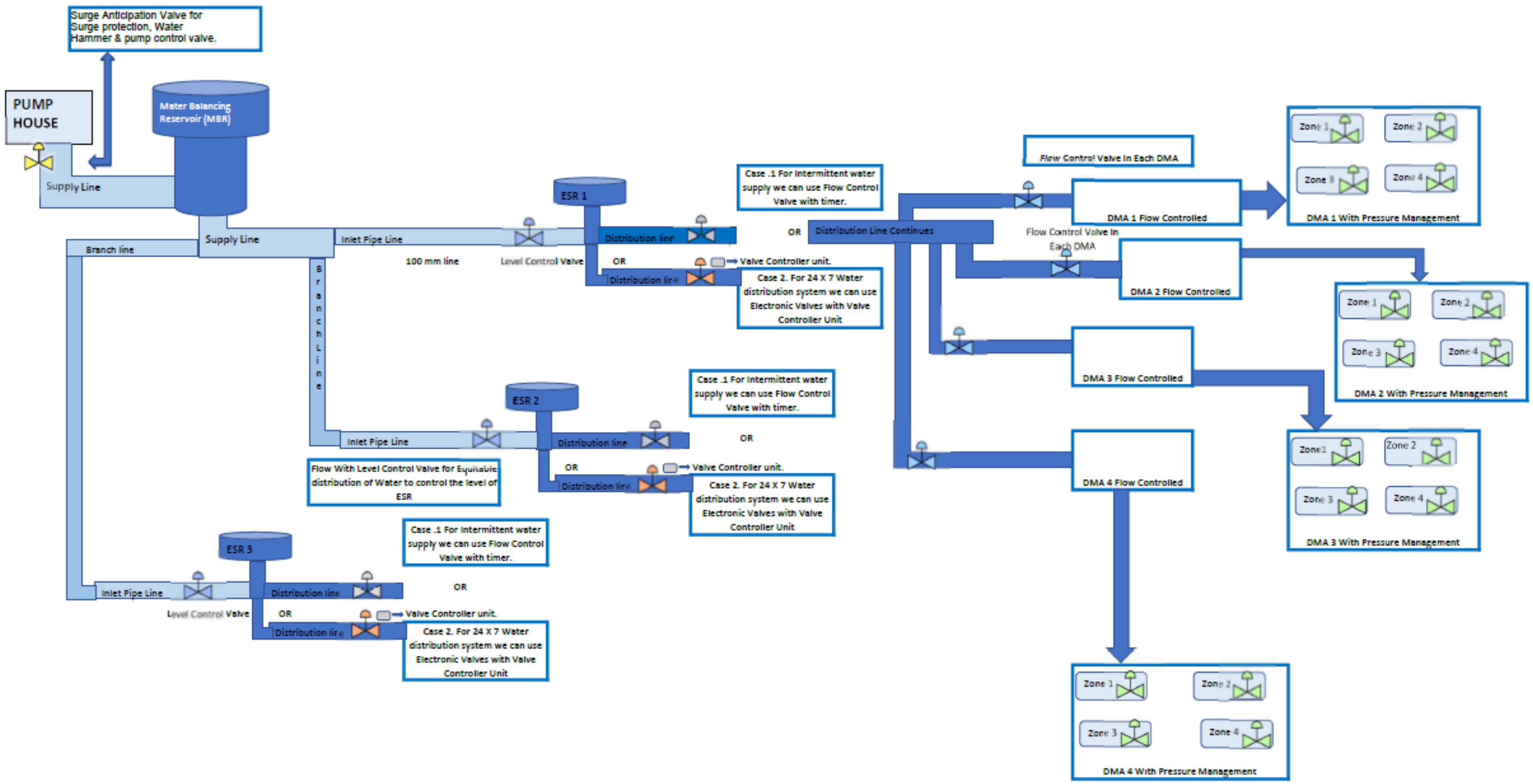


Figure 11.123: City is Huge and there are Multiple Sub-Zones in Each DMA

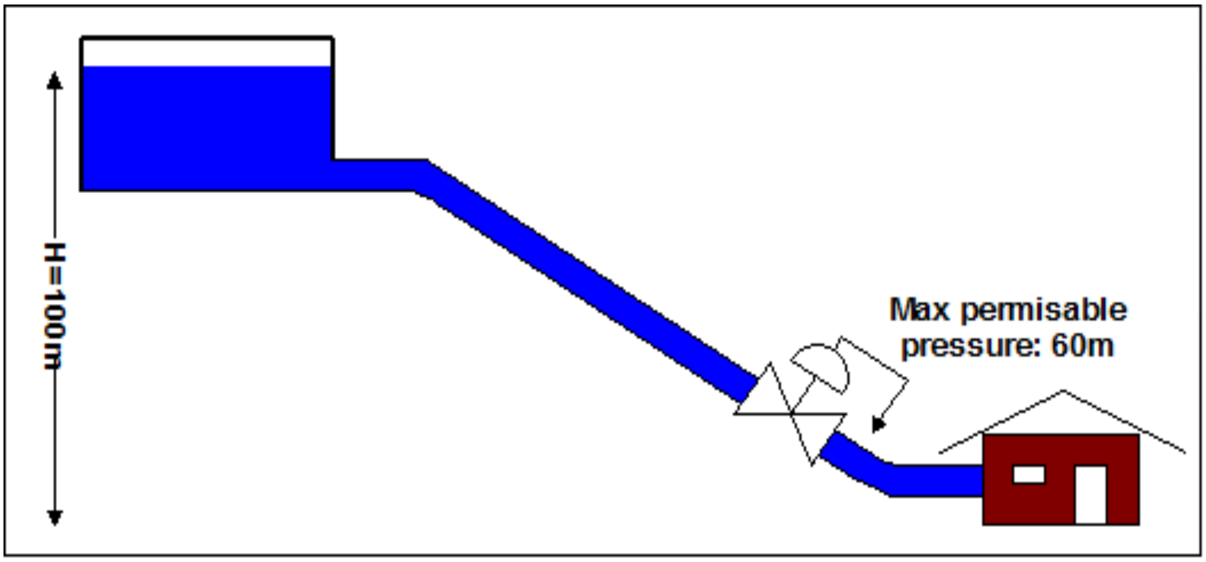


Figure 11.124: Demand Catered in Hilly Areas

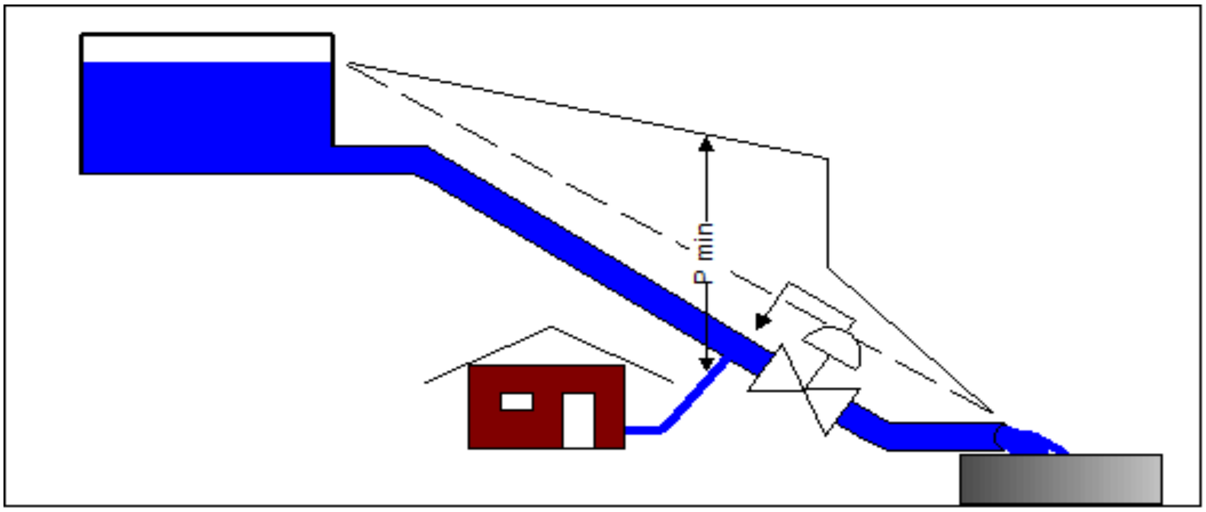


Figure 11.125: Maintenance of Certain Minimum Pressure in the Pipeline

11.23.1.15 Foot Valve

Foot valves for water works purposes are covered under IS 4038 (1986), Reaffirmed 2020: Foot valves for water works purposes [CED3: Sanitary Appliances and Water Fittings]. This standard covers requirement for flanged and screwed end foot valves of both swing and lift type for use with centrifugal pumps for water works purposes. It covers screwed end valves from 25 mm to 150 mm nominal size and flanged end valves from 50 mm to 450 mm nominal sizes.

11.23.1.16 Pressure Reducing Valves

These are used to automatically maintain a reduced pressure within reasonable limits in the downstream side of the pipeline. This type of valve is always in movement and requires scheduled maintenance on a regular basis. This work is facilitated if the valve is fitted on a bypass with isolating valves to permit work to proceed without taking the main out of service. If the pressure reducing valve is fitted on the main pipeline, a bypass can be provided for emergency use. Needle-type valves which can be hydraulically controlled or motor operated with a pressure regulator are used for large aqueduct mains.

11.23.1.17 Pressure Sustaining Valves

Pressure sustaining valves are similar in design and construction to pressure reducing valves and are used to maintain automatically the pressure on the upstream side of the pipeline.

Spacing of Valves and Interconnections

The pipeline should be divided into sections by valves to avoid the necessity of emptying the whole pipeline in case of repair, each section being provided with an air valve and scouring facilities. The need for scour should be particularly borne in mind when the layout of the pipeline and siting of the valves are finalised, as they cannot always be arranged in the best position due to likely difficulty in disposing of the discharge. They are necessary for scouring the mains and, hence, should be in proportion to the size of the main.

It is desirable to have valves close together in more densely built-up areas. Ease of access to the valves is also important as the time taken in shutting of a valve in an emergency may be mostly spent in reaching it. In gravity mains, automatic valves, self-closing if a pipe bursts, may also be provided for protection to property as well as to prevent excessive wastage of water.

Where there is more than one pipeline, they should be interconnected at each site of main valves, so that only shortest possible length of one pipeline need be put out of commission at a time. The interconnection will entail only negligible loss of head if its area is not less than two-thirds that of the largest main.

Also, when two or more mains are connected in parallel, the scours may be interconnected so that either main can be refilled from the other while the master valve is shut. Charging through a scour can be done speedily with less risk than charging over a summit, the danger of surging from trapped air being much reduced.

Integrated Valves (Modulated Valve)

Integrated or modulated valves combine the functionality of a flow control valve (FCV), flowmeters and a pressure reducing valve (PRV). They help maintain a constant flow rate and pressure, which is essential for the system's proper functioning. These valves are commonly used in flow metering applications where a constant flow rate needs to be maintained despite changes in upstream pressure.

The FCV component of the integrated valve regulates the flow rate, while the PRV component maintains a constant downstream pressure by reducing the pressure as required. This allows for accurate flow rate measurement even in situations where the upstream pressure fluctuates.

Proper selection, installation, and maintenance of these valves are essential for the reliable and efficient functioning of the system. Some guidelines for the use of integrated valves in water supply systems are as follows:

1. Determine the required flow rate and pressure: Before selecting an integrated valve, it is important to determine the required flow rate and pressure of the system. This will ensure that the selected valve can handle the required flow rate and pressure range.

2. Consider the valve size: The valve size should be selected based on the size of the pipe in the system. A valve that is too small will cause pressure drops and flow restrictions, while a valve that is too large will be inefficient and expensive.

3. Select the appropriate valve type: There are different types of integrated valves, such as globe valves, butterfly valves, and ball valves. The valve type should be selected based on the application and the required level of flow control and pressure regulation.

4. Install the valve correctly: The valve should be installed in the correct orientation and with the appropriate fittings and gaskets. The installation should be done in accordance with the manufacturer's instructions to ensure proper functioning of the valve.

11.23.2 Manholes/Inspection and Repair Chamber

Manholes are provided at suitable intervals along the pipeline. They are helpful during construction and later on serve for inspection and repairs. These are usually spaced 300 to 600 m apart on large pipelines. Their most useful positions are at summits and downstream of main valves. They are commonly provided in the case of steel and concrete pipelines and are less common in the case of cast iron and asbestos cement pipelines.

Surface boxes, Guards and underground chambers for the purpose of utilities such as Inlet chamber of DMA etc. shall be as per BS 5834-2-2011 (91.140.60 Water Supply Systems). The surface box for sluice valves shall be as per BIS code IS 3950:1979.

11.23.3 Fire Hydrants

Indian Standard Provision and Maintenance of Water Supplies for Fire Fighting – Code of Practice to be followed.

The following BIS codes are to be followed with regard to selection and installation, etc. for fire hydrants:

(a) IS 908 (1975, Reaffirmed 2020) Fire Hydrant stand post type

(b) IS 884 (1985, Reaffirmed 2020) First-aid hose reel for fire hydrant

(c) IS 8442 (2008, Reaffirmed 2018) Stand post type water and foam monitor for fire fighting

(d) IS 13039: (2014, Reaffirmed 2019) Indian Standard External Hydrant Systems – Provision and Maintenance – Code of Practice

(e) IS 3844 (1989, Reaffirmed 2020): Code of Practice for Installation and Maintenance of Internal Fire Hydrants and Hose Reels on Premises – Fire Safe World

(f) IS 2190 (2010, Reaffirmed 2020): ‘Code of practice for selection, installation and maintenance of portable first-aid fire extinguishers (second revision).’ This standard covers requirement in respect of installation and maintenance of internal fire hydrants and hose reel systems with or without sprinkler installation for different types of buildings. Internal fire hydrants are intended for use by fire brigade or other trained personnel and provide means of delivering considerable quantities of water to extinguish or to prevent the spread of fire. A fire hydrant is an outlet provided in a pipeline for tapping water mainly for the purpose of firefighting (or fire extinguishing). However, sometimes these may also be used for withdrawing water for certain other purposes such as sprinkling on roads, flushing streets, etc. When a fire breaks out, water is obtained for firefighting from a nearby fire hydrant through a fire hose. For firefighting, usually large quantity of water at high pressure is required in order to make it to reach to the place of occurrence of fire. Thus, if water at required pressure is available from a fire hydrant, it can be directly used for firefighting through a fire hose connected to the outlet of the fire hydrant. However, if water at much higher pressure is required, the same is developed by attaching a fire engine or a pump to the fire hydrant outlet. The fire engine or the pump draws water from the fire hydrant, boosts its pressure, and the high pressure water coming out from the outlet of the fire engine, or the pump is used for firefighting through a fire hose connected to the outlet of the fire engine or the pump. At the end of the fire hose, a nozzle is provided to develop a powerful jet of water. The number of fire hydrants in a distribution system and their location depends on various factors such as chances of fire occurrence, requirement of water for firefighting, utility of buildings, population of area, etc. Generally, fire hydrants are placed at all important road junctions and at intervals not exceeding about 300 m.

In case of industries of high hazard category (Gr G-3, H, and J), the hydrants should be installed at every 30 m apart along building line and the hydrant outlet should be single or double hydrant with provision of landing valves.

11.23.4 Water Metres

Water metres are the devices which are installed in pipelines to measure the quantity of water flowing through them. The water flowing through pipelines is supplied to various consumers for domestic, industrial, and commercial uses and its measurement is necessary to charge the consumers according to the quantity of water supplied to them. A separate chapter on water metres has been included in this Manual Part A, Chapter 13 - Water Metres.

11.24 24×7 Water Supply and Selection of Pipe Materials and Pipe Appurtenances

Continuous pressurised water supply is a solution for improving deteriorating quality problem in the country. Through the leaks in intermittent water supply, outside contaminants enter pipeline during non-peak hours owing to the vacuum that is developed inside pipeline. Thus, water becomes non-potable. On contrary, in 24×7 system due to pressure inside pipeline, outside impurities can't enter in and quality of water remains intact. Life of distribution networks increases as steady pressure in the pipes causes less damage to the pipes. A better demand management is possible due to elaborate metering and effective leakage control. It also results in less storage of water or none at all, which in turn reduces wastage of water. Continuous supply of clear water (contamination free) boosts the economy and attracts more industries and businesses.

It is now well established that intermittent water supply leads to health risks for users due to the higher likelihood of contamination of water pipelines through joints and damaged segments during periods when the system is not pressurised. The absence of sound technical and managerial systems associated with intermittent supply makes supply and demand management extremely difficult. This prevents effective estimation or control of the amount of water produced, transmitted, and distributed. Intermittent supply of water also causes great inconvenience to households, especially to women and children who most often bear the brunt of the hardship associated with inadequate and unreliable water supply. To cope with these shortcomings, customers revert to expensive coping strategies such as building expensive underground sumps and overhead tanks, and installing booster pumps, treatment devices, etc. In all ULB's, the NRW level is high due to improper laying, jointing, and O&M of the distribution system. Thus, the engineer should put a lot of emphasis on selection of pipe and compatible specials and fittings, while designing and planning the water supply system.

CHAPTER 12: SERVICE RESERVOIRS & DISTRIBUTION SYSTEM

12.1 Introduction

The purpose of the distribution system is to deliver wholesome water to the consumers at their location in sufficient quantity at adequate residual pressure with "Drink from Tap" facility. Consumers of potable water include households, hospitals, restaurants, and public amenities, etc., who rely on the safe treated water supply for drinking, bathing, cooking, and gardening, among other things. The customers and the nature in which they use water are the driving mechanism behind how the water distribution behaves. Water use can vary over time both the long term (seasonal) and the short term (daily) and over space. Good knowledge of how water demand is distributed across the system is essential for accurate planning of the system.

The water can be distributed either by gravity feed system using the service reservoirs or by direct feeding with pumps (especially VFD pumps).

12.2 Basic Requirements

The basic requirements of a good and sound distribution system are to supply potable water to all consumers in the requisite quantity and pressure, as well as prevent the contamination of water. The distribution system should be capable to cater the need even during emergencies like firefighting. The leakage in the system should be minimum within the permissible limit.

The requirement of an ideal water distribution system can be considered as:

a) The system should be able to supply potable water to all intended places with enough pressure during normal and abnormal conditions such as those arising due to failure of any pipe, pump or other components, excessive demand due to fire, or other purposes at all times.
b) Water quality should not deteriorate while in distribution pipe, the layout should be such that not more than 250 consumers in plain area and 80 consumers in hilly area are affected during repair of any site of the system.
c) All distribution pipes should be laid at least one metre above the sewer lines.
d) Joints should be watertight to keep losses (due to leakage) to bare minimum.

12.2.1 Continuous Versus Intermittent System of Supply

In the continuous system of supply, water is supplied to consumer throughout the day, whereas in the intermittent system, the consumer gets supply only for certain hours. The intermittent system has major disadvantages of contamination, high Non-Revenue Water (NRW), inequitable distribution of water supply and high coping cost. The distribution system is usually designed as a continuous system but often operated as an intermittent system. There is always a constant doubt about the supply hours in the minds of the consumers which leads to limited use of water supplied and does not promote personal hygiene. The water stored during non-supply hours in different containers/vessels may get contaminated and once the supply is resumed, this water is wasted, and fresh supply stored. During non-supply hours, polluted water may reach the water mains through leaky joints and consumer's underground storage tanks and thus could pollute the protected water. There will be difficulty in finding sufficient water for firefighting purposes also during these hours. The taps are always kept open in such system leading to wastage when supply is resumed. This system does not promote hygiene and hence, intermittent supply should be discouraged.

However, to avoid wastage and to enjoy a 24×7 system in case of intermittent supply, consumers usually construct an underground sump with pumping system to lift the water from sump to overhead tank, and a household treatment system. Considering this extra cost, such type of continuous system is not desirable.

24×7 continuous water supply is achieved when water is delivered continuously to every consumer residing in the service area throughout the day, every day of the year through a distribution system that is continuously full and under positive pressure which results in supply of fresh water, with no chance of contamination and requirement of comparatively lesser size of pipes in distribution system. Therefore, the continuous system is always preferred especially for "Drink from Tap" water supply.

12.2.2 System Pattern

Distribution system can be either looped or branched as shown in Figure 12.1. As the name implies, in a looped system, water can take multiple courses from the source to a specific consumer, whereas in a branched system, also known as a *tree* or *dendritic* system, water can only take one path from the source to the customer. A looped system is preferable over a branched system because, when combined with enough operational valves, it can give an additional level of reliability. Consider the case of a main break near the reservoir. That break can be isolated and fixed in the looped system with minimal impact on consumers outside of the immediate vicinity. Consumers downstream from the failure point in the branching system, on the other hand, will have their water supply cut until the repairs are completed. Another advantage of a looped system is that the velocities will be low and system capacity will be greater because there is more than one path for water to reach the user. At the instance of fire outbreak, in loop system, water can be made available from alternate sides of the pipeline network.

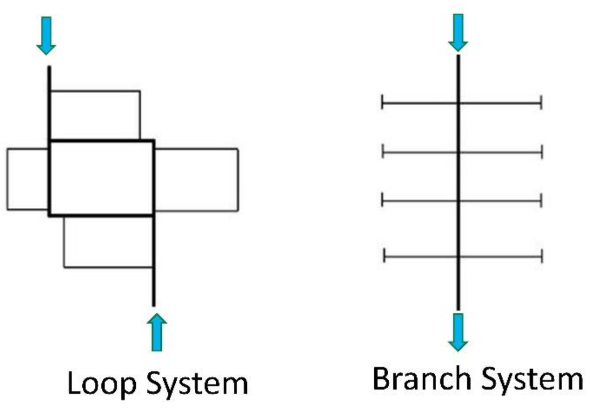


Figure 12.1: Pattern of Distribution System

12.2.3 Condition Assessment and Integration of Existing Network

Details are mentioned in Chapter 2 "planning and investigation" in Part A of this manual.

12.2.4 Layout of the Network

Layout of system is important from pressure control point of view. Usually, branched type of systems is observed in old networks. However, such networks have their own disadvantages and in urban city usually a combination of branched and looped network is provided. Loops in the network help in maintaining reliable and more equitable pressure. Therefore, the layout should be such that main pipelines form loops. Direct consumer connections through these mains should be avoided. Secondary (rider pipeline) smaller diameter (80-100mm internal diameter) branched networks originating at different points from this main loop network should be used for providing service connection. This branched network, if possible, should follow topography of the ground.

The distribution layout should be such that it facilitates the hydraulic isolation of sections, and metering for assessment/control of leakage and wastage. Distribution pipes are generally laid below the road pavements (followed by prompt reinstatement of road after testing) and as such, their layouts generally follow the layouts of roads.

Operational zone (OZ) is the division of a project area into smooth manageable areas. OZ in the distribution system ensures equalisation of supply of water throughout the area. The zoning depends upon (a) location of service area, (b) density of population, (c) type of locality, **(d)** topography, (e) facility for isolating for assessment of waste and leak detection, (f) type of activity, and (g) physical barrier like expressway, canal river, railway track. The OZ is such that each one is served by a separate service reservoir.

12.2.5 Pressure Zones

Initially, the GIS-based contours should be generated using suitable survey method. Using GIS technique, Inverse Distance Weighted (IDW) polygons or *Topo-to-Raster* polygons shall be formed as described in Chapter 2. Different elevation polygons shall be demarcated with colour code in GIS. This will help to plan high elevation as well as low elevation OZ in the distribution system. If there is an elevation difference more than 20-25m, then the OZs can be designed accordingly. The reservoirs of neighbouring zones may be interconnected through feeder main/transmission main to provide emergency supplies. The layout should be such that the difference in pressure between different areas of the same zone or same system does not exceed 5 m.

Layout in Hilly Areas

Pressure management in hilly areas is most challenging as the service reservoir is located on the higher level on the hill and there may be a large elevation difference between the service reservoir and the consumer location at the bottom or slope of the hill. Proper OZs/DMA is essential in this case. OZs (area served by a service reservoir) must be separated and pressure reducing valves (PRVs) and break pressure tanks should be provided at appropriate locations to manage excessive pressure. It is illustrated below by taking example of the hilly city. Three pressure zones at hilly city are shown in Figure 12.2

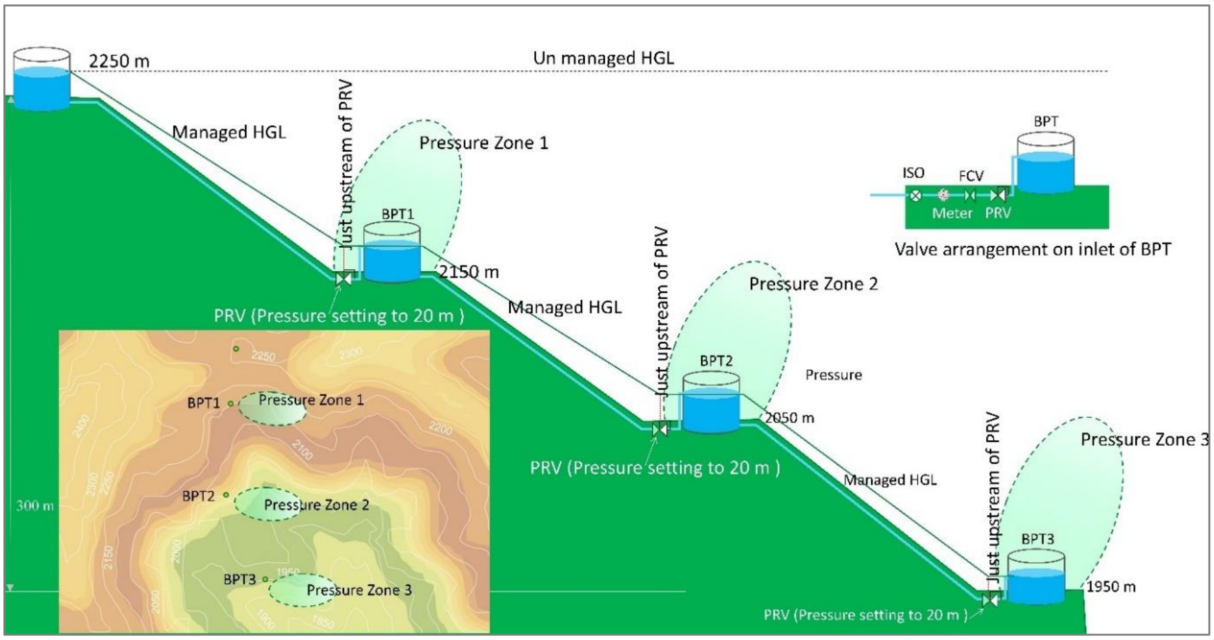


Figure 12.2: Pressure zones in hilly city

Pressure zones at different ground elevations can be formed using GIS technology as discussed in Chapter 2. Break pressure tanks- BPT1, BPT2 and BPT3 also serve the function of service reservoir for pressure zones 1, 2, and 3, respectively. Before each of BPT, a PRV is installed, which is set to limit the zone pressure to 21 m. The valve arrangement on inlet of each BPT is shown in Figure 12.2. Thus, the residents in each pressure zone get water with required residual nodal pressure of 17-21 m. Direct acting PRVs can be used to limit pressures up to 17-21 m.

12.2.6 Location of Service Reservoirs

The location of service reservoirs is of importance for regulation of pressures in the distribution system as well as for coping up with fluctuating demands. In a distribution system fed by a single reservoir, the ideal location is a central place at higher elevation in the distribution system, which effects maximum economy on pipe sizes. Where the system is fed by direct pumping as well as through reservoirs, the location of the reservoirs may be at the tail end of the system. If topography permits, ground level reservoirs may be located taking full advantage of differences in elevation. Even when the system is fed by a central reservoir, it may be desirable to have tail end reservoirs for the more distant districts. These tail end reservoirs may be fed by direct supply during lean hours or booster facilities may be provided.

12.3 General Design Guidelines

Water distribution network should be designed to meet peak hourly demand at required pressure.

12.3.1 Elevation of Reservoir

The elevation of the service reservoir should be such as to maintain the minimum residual pressure in the distribution system consistent with its cost effectiveness. The Lowest Supply Level (LSL) of water in Elevated Service Reservoir (ESR) shall approximately be equal to highest ground level in OZ + required minimum residual head + frictional losses.

A suitable combination of pipe sizes and staging height has to be determined for optimisation of the system. With concept of decentralised planning, higher head loss gradient can be achieved. Presently, 10 to 12 storey buildings are quite common and construction technology is quite modernised, hence, the ESR could be provided with high staging height. However, the maximum staging height of an ESR should be properly designed.

Equitable distribution of water with designated pressures in 24×7 continuous water supply is achieved by whole-to-part approach with two stages, namely: (a) First stage for equitable distribution from master balancing reservoir (MBR) to ESRs; and (b) second stage for equitable distribution from ESRs to DMAs. Equalisation of pressures (residual heads) at full supply level (FSL) of service tanks helps in effective and equitable supply of water to various ESRs in city by the transmission mains. In first stage, the MBR supplies water to different ESRs. Inlet of each tank should be provided with isolation valve, bulk meter and then flow control valve (FCV)/PRV. Normally, the FCV is sufficient because while controlling the flow, the pressure is proportionally reduced. However, in hilly area or the area having steep slopes, both the PRV and FCV are required. In such situation, the isolation valve, bulk meter, PRV, and FCV are provided in sequence in the direction of flow. The FCV is set such that the inflow to the tank would be as per demand of the OZ served by the tank. The control valve with level controller should be installed on the inlet of the tank. The control valve with level controller ensures that the tank does not overflow, and this eliminates the physical losses due to overflow of tanks.

12.3.2 Boosting

For 24×7 continuous water supply system, online boosting with variable frequency drive (VFD) pump at following locations in the OZ of existing ESRs can be provided for achieving required minimum nodal pressures for continuous water supply system:

- Online boosting at the outlet of tank for entire OZ having residual nodal pressures less than 17-21 m or 12-15 m (as the case may be).
- Online boosting for the branch line to area having residual nodal pressures less than 17-21 m or 12-15 m. (as the case may be).

If any of the DMAs is found to be heavily leaking due to high pressure, then RPM of VFD can be controlled by a suitable frequency. This can also be adjusted using PRV/normal throttling valve to regulate the pressure.

The details of direct feeding by VFD pumps are mentioned in Chapter 5 “Pumping Stations and Pumping Machinery” in Part A of this manual. Hydraulic model using VFD is described in **Annexure 12.4**.

12.3.3 Location of Mains

For concrete roads wider than 6 metres, the distribution pipes may be provided on both sides of the road, by running rider mains suitably linked with trunk mains. Pipes on both side of the road shall be so planned that they form boundary of OZ/DMA.

12.3.4 Valves and Appurtenance

Various types of valves are required in distribution network to control flow and pressure. Also, to remove the suspended/deposited particles/incrustation. These have been discussed in Chapter 11.

12.3.5 Locations for filling Fire Brigade

The fire brigades can be filled by water at the outlet of service tank. For this purpose, 150 mm diameter pipe can be taken out as an offset from the main outlet pipe. This can be operated at the time of filling the fire brigade tanks.

12.4 Service Reservoirs

12.4.1 Function

A service reservoir has the following main functions:

- To act as a buffer storage and balance the fluctuating demand (peak rates of demand) of the distribution system.
- To provide storage for firefighting.
- To ensure suitable residual pressure to the distribution system and minimise the pressure fluctuations and provide water supply even during instance of power failure.
- To allow pump and treatment plant to operate at constant flow and head.

12.4.2 Capacity

The minimum service or balancing capacity depends on the hours and rate of pumping in a day, the probable variation of demand or consumption over a day. The minimum balancing capacity can be calculated from a mass balance diagram. The variation of demand in a day for a town which depends on the supply hours may have to be assumed or known from similar towns or determined based on household survey.

Balancing capacity of the service reservoir in urban and rural areas shall be as per Table 2.7 of the Chapter 2 of this Part A manual. Additional storage should be provided for firefighting demand.

An illustrative example is provided in **Annexure 12.1** to show design calculations for obtaining minimum reservoir capacity of service reservoir using mass balance method.

12.4.3 Structure

The elevated reservoirs are used principally as distributing reservoirs and can have shapes like circular, square, rectangular, and conical or may be of Intze type and any other shape. The ground level reservoir is generally preferred as storage reservoir if a suitable higher ground level is available. Service reservoir can be circular or square or rectangular in shape. If it is circular, it is usually constructed of RCC and in the case of other shapes, it is constructed either of RCC or masonry. These are covered under in IS 3370 (Part 1-4). Small capacity tanks can be fabricated with steel or PVC or HDPE. Circular shapes are generally preferable as the length of the wall for a given capacity is a minimum and further the wall itself is self-supporting and does not require counterfort. Reservoirs of one compartment are generally square and those of two or three compartments may be rectangular with length equal to one and half times the breadth. The economical water depth for reservoirs is mentioned in Chapter 2 in part A of this manual. The service reservoirs should be covered to avoid contamination and prevent algal growths. Suitable provision should be made for air vents, manholes, mosquito-proof ventilation, access ladders, scour and overflow arrangements, water level indicator, and if found necessary, the lightning arresters.

12.4.4 Inlets and Outlets

The outlet/ draw pipe should be placed 15 centimetres above the floor and is usually provided with a strainer of perforated cast iron or a bell mouth. The reservoirs filled by gravity are provided with sluice/ball valves of the equilibrium or other type which close when water reaches full tank level. The overflow and scour main should be of sufficient size to take away by gravity the maximum flow that can be delivered through the reservoir. The sizes of inlet and outlet shall be computed considering the velocity as 1 m/s. The material of these pipes shall be metallic. The outlet of the scour and overflow mains should be protected against the entry of vermin and from other sources of contamination. The inlet or outlet of reservoir should be such that no water stagnates.

Box 1: Speeding up Construction Activity

To speed up the construction activity and to deliver the water to the consumer on priority, designer can opt for speedy construction methods like precast RCC staging with prefabricated metal container.

Below mentioned are the specifications which needs to be followed for precast RCC Staging:

1) Minimum grade of concrete for precast members shall be M40
2) Reinforcement bars shall be of high yield strength deformed bars Fe 500TMT confirming to IS1786 – Latest Revision
3) Strength of precast element at the time of demolding shall be a minimum 15 Mpa
4) Connection between precast elements can be with coupler filled with non-shrink grout or with bolted connection
5) Properties of grout to be filled between precast elements and coupler is:
    - Compressive strength at 6 hrs. is >15 N/mm^2
    - Compressive strength at 24 hrs. is >30 N/mm^2
    - Compressive strength at 28 days is >65 N/mm^2
6) Appropriate seismic zone and wind speed is to be considered for the design based on the location

Codes & Standards: The above design shall refer to the following Codes (Latest Revision)

1) IS: 456 -2000 – Code of practice for plain and reinforced concrete
2) IS: 875 -2015 – Code of practice for Design Loads for Building and Structures (Part-1 to 5)
3) IS: 1893-2016 – Criteria for Earthquake Resistant design of structures, Part-1 General Provisions of buildings
4) IS: 1893-2014 - Criteria for Earthquake Resistant design of structures, Part-2 Liquid Retaining Tanks
5) SP: 34 - Handbook on Concrete Reinforcement and Detailing
6) IS 11682:1985 – Criteria for Design of RCC Staging for Overhead Water Tanks

Below mentioned are the specifications which needs to be followed for Prefabricated Metal Container:

1) Container design and standards shall be in accordance with AS2304.
2) Minimum design life of prefabricated metal container shall be of 25 years.
3) Base material of wall shell shall be of steel with minimum yield strength of 300 MPa.
4) Base material of Roof sheet shall be of Steel with minimum yield strength of 550 MPa.
5) Composition of coating on the base material i.e., Zn, Al, and Si shall be 43.5%, 55%, and 1.5% respectively.
6) The tank wall shall be stiffened so that it will not buckle from wind action while it is in an empty state.
7) The minimum base metal thickness for tank wall shell shall be 0.8 mm.
8) The tank may be stiffened by increasing the panel thickness, profiling, the use of laminations or the installation of circumferential stiffeners as described in AS2304.

9) Zn-Al corrugated steel tank is anchored to the base slab with bolts and stiffener arrangement to hold the water inside the container; food-grade reinforced PVC liner of minimum 890 GSM thickness shall be used.
10) Serviceability of liner shall be for a range of external temperatures from -5 °C to +70 °C.

When there are two or more compartments, each compartment should have separate inlet and outlet arrangements, while the scour and overflow from each compartment may be connected to a single line. To avoid waste of energy, it is advantageous to form the opening of the outlet with a configuration identical to the surface. This could be achieved by providing a bell mouth at the opening of the outlet pipe.

12.5 Floating Reservoirs/Tanks

A tank is said to be "floating on the distribution network" when connected by a single pipe used both as inlet and outlet pipe. When the rate of supply from main reservoir exceeds the demand of consumers, water is received by the floating reservoir/tank. On the other hand, when consumer demand exceeds supply from main tank, water is supplied by the floating tank through the same pipe. The relation between rate of supply, rate of demand, and tank capacity is based on a study of the service required as in case of service reservoirs.

12.6 Hydraulic Network Analysis

12.6.1 Principles

In interconnected networks of hydraulic elements, every element is influenced by each of its neighbour. The entire system is interrelated in such a way that the condition of one element must be consistent with all other elements. As discussed earlier in Chapter 6, head loss and flow relationship in a pipe is non-linear, and for the pressurised flow in pipe, either Darcy-Weisbach or Hazen-Williams equation (Chapter 6) can be used. The pipe head loss relationship in a general form can be written as

$$h = R\,Q^n \qquad \text{(Eq. 12.1)}$$

where, h is head loss in pipe, Q is discharge in pipe, and R is resistance of the pipe, n is exponent of discharge in Darcy-Weisbach/Hazen-Williams equation.

Two basic relationships, also known as Kirchoff's law, governing flow distribution in a network under steady state condition are:

a) node flow continuity relationship; and

b) path and loop head loss relationship.

A. Node Flow Continuity Equation

The principle of conservation of mass dictates that the fluid mass entering at a node or junction will be equal to the mass leaving that node or junction. Mathematically, it can be expressed as

$$\sum_{x\epsilon j} Q_x - q_j \ = 0 \qquad \text{(Eq. 12.2)}$$

where, Q_x = flow in pipe x, and q_j = water demand at node j. Inflows at a node are considered positive and outflows are considered negative in Eq. (12.2).

In modelling, when extended period of simulations are considered water can be stored and withdrawn from the tanks, thus a term is needed to describe the accumulation of water in certain nodes

$$\sum_{x\epsilon j} Q_x - q_j \pm \frac{dS}{dt} = 0 \qquad \text{(Eq. 12.3)}$$

where dS/dt = change in storage.

The conservation of mass equation is applied to all junction nodes and tanks in a network and one equation is written for each of them.

B. Path and Loop Head Loss Equations

A part of the energy possessed by flowing fluid is lost to maintain the flow. Thus, the difference of energy at any two points connected in a network is equal to the energy gains from pumps and energy losses in pipes and fittings that occur in the path between them. This equation can be written for any open path between any two points or paths around loops. For the closed loop, the algebraic summation will be zero.

$$\sum_{x\epsilon l} h_x = 0 \qquad \text{(Eq. 12.4)}$$

Where, h_x = head loss in pipe x. Head gains in the path are considered positive and head drops are considered negative in Eq. (12.4).

12.6.2 Methods for Network Analysis

A general problem of network analysis consists of determining the pipe flows and nodal pressures for a real water distribution network under the condition of known demands. For a single source branched network, the analysis can be carried out by starting from any dead ends and determining flows in the connected pipe of each dead end using Eq. 12.2. With the known pipe flows in some of the pipes, nodes with one unknown pipe flows are selected and applying Eq. 12.2, flows are calculated. The process is continued till the source node is reached, thereby flows in all the pipes are known. Using these pipe flows, head loss in each pipe is obtained using Eq. 12.1. Now, with the known HGL at the source node and the head loss values, HGL at demand nodes are obtained and residual pressures are calculated. However, analysis of muti-source branched network and looped networks are not that simple. Analysis of looped networks and muti-source branched networks generally requires formation of required numbers of independent equations either in terms of nodal flows or nodal heads by using Eqs. (12.1) to (12.4). Some equations, if not all, are non-linear and iterative procedure is used for their solution. Several methods are available for analysis of WDNs. The commonly used methods are the Hardy Cross Method, Newton-Raphson Method, Linear Theory Method, and the Gradient Method. Brief description of these methods and an illustrative example is provided in **Annexure 12.5**.

12.6.3 Types of Analysis

A. Node Head Analysis and Node Flow Analysis or Pressure-Dependent Analysis (PDA)

A simple type of analysis is carried out by assuming that the nodal demands are satisfied. Therefore, outflows at all demand nodes are considered equal to required demand. This type of analysis is useful in checking the adequacy of the network in meeting the design demands. As demands are assumed satisfied and corresponding nodal heads are calculated; it is called node head analysis (NHA) or demand dependent analysis (DDA). If the pressures at all the nodes are found above minimum required pressure, the network performance is considered satisfactory, else not satisfactory. Even though the DDA shows unsatisfactory performance through deficiency in pressure, it is found not capable of predicting actual deficient nodes as the calculated pressure deficiency is corresponding to the assumption of meeting demands. Usually, while designing a network, some modifications in component sizes are made to make it satisfactory. However, in the absence of such modifications like pipe or pump failure condition or excessive demand such as fire demand, the water will be available fully at some of the nodes, partially at some and not at all at some of the nodes. Thus, available flows at a node depends on the available pressure. Therefore, there exists a relationship between available flows and available heads called node-head-flow-relationship (NHFR). The network analysis to obtain available nodal flows considering available pressure is called node flow analysis (NFA) or pressure-dependent analysis (PDA). This type of analysis is useful in reliability analysis as well as in optimal network design methodologies.

A simple NHFR was suggested by Bhave (1981) in which at available pressure above some minimum pressure, nodal demand is considered completely satisfied. At available pressure below some minimum head, no outflow was considered; at available pressure just equal to the minimum head, outflows are considered between 0 and required demand, and obtained using optimisation. Several other relationships are available in the literature. Wagner et al. (1988) and Chandapillai (1991) suggested a NHFR defined by two heads - minimum and desirable heads. Full demand is considered met at available pressure above desirable pressure. Partial flow using a parabolic relationship in case available head is in between some minimum and desirable pressure, and no flow if available pressure at a node is below minimum pressure head. As the demands of several consumers have different pressure requirements due to their locations, NHFR of Wagner et al (1988) is more appropriate than of Bhave (1981). Node-head-flow Relationships are shown in Figure 1.

As suggested by Bhave (1981), the available flow at demand node *j* may be characterised as follows:

$$q_j^{\mathrm{avl}} = q_j^{\mathrm{req}} \text{ (adequate flow), if } H_j^{\mathrm{avl}} > H_j^{\min} \tag{12.5}$$

$$0 \le q_j^{\mathrm{avl}} \le q_j^{\mathrm{req}} \text{ (no flow, partial flow or adequate flow), if } H_j^{\mathrm{avl}} = H_j^{\min} \tag{12.6}$$

$$q_j^{\mathrm{avl}} = 0 \text{ (no flow), if } H_j^{\mathrm{avl}} < H_j^{\min} \tag{12.7}$$

in which H_j^{avl} is the available head at demand node *j*; q^{avl} = available flow, q^{req} = required flow; and H_j^{min} is the minimum required head.

Parabolic NHFR for HGL values between $H^{\min}$ and H^{des} as suggested by Wagner et al. (1988) and Chandapillai (1991) is as follows:

$$q_j^{\mathrm{avl}} = q_j^{\mathrm{req}} \left(\frac{H_j^{\mathrm{avl}} - H_j^{\min}}{H_j^{\mathrm{des}} - H_j^{\min}} \right)^{\frac{1}{n_j}}, \text{ if } H_j^{\min} < H_j^{\mathrm{avl}} < H_j^{\mathrm{des}} \tag{12.8}$$

where n_j is a coefficient. Different values between 1 and 2 have been suggested in the literature (Bhave et al., 2006).

NFA/PDA is found better not only to predict nodes having deficiency in pressure but also in quantifying the flow deficiency at those nodes. Such information is useful when authority needs to prioritise the areas most affected due to the failure of any component and make proper decision regarding water supply through tankers in affected zone. Also, the method is found useful in problems related to reliability analysis, optimal network design using evolutionary techniques to calculate penalties, flushing of contaminated water, pressure-dependent leakage analysis. The new version of EPANET 2.2 has the facility of PDA using Wagner's parabolic relationship.

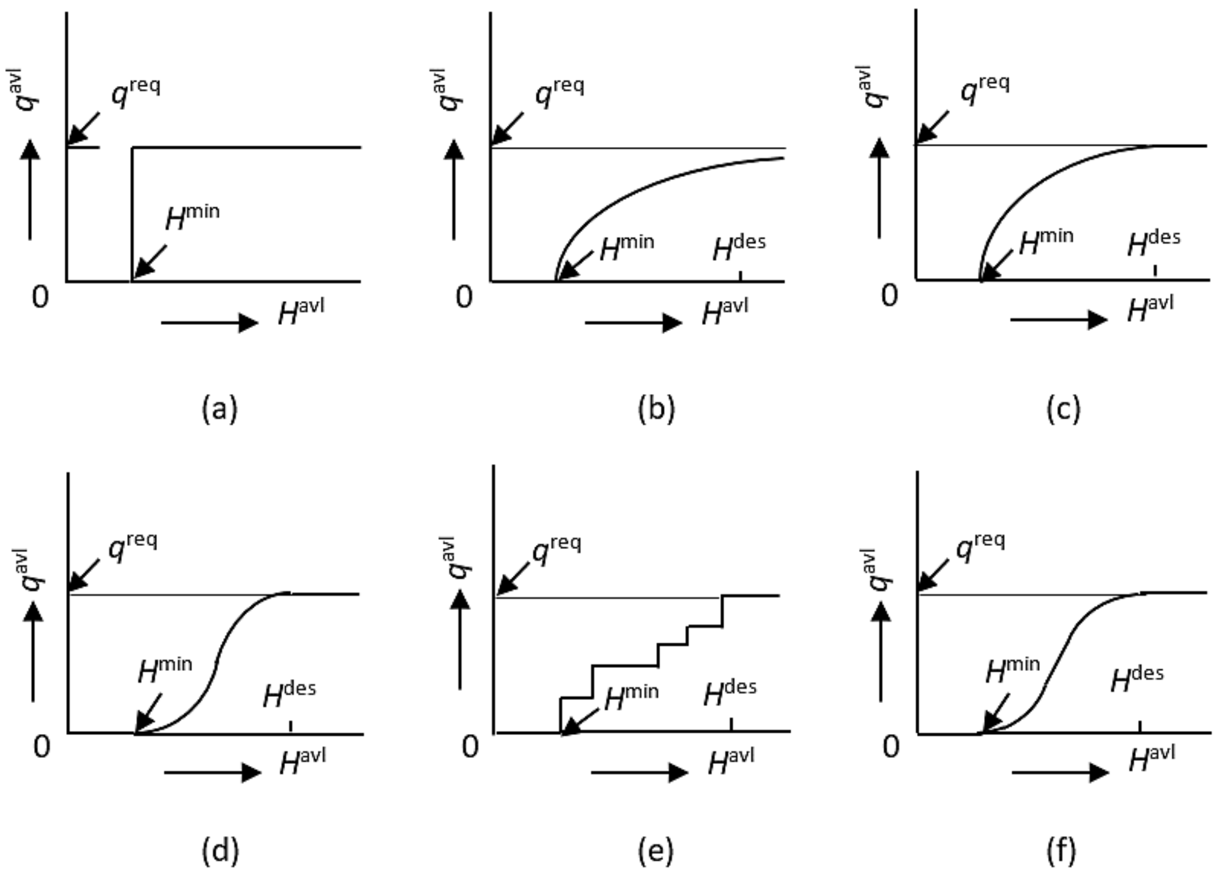


Figure 1. Node-head-flow Relationships: (a) Bhave (1981), (b) Germanopoulos (1985), (c) Wagner et al. (1988), (d) Fujiwara & Ganesharajah (1993), (e) Kalungi and Tanyimboh (2003), (f) Tanyimboh and Templeman (2010)

B Dynamic Analysis or Extended Period Simulation

The NHA (or DDA) and NFA (or PDA) gives instantaneous picture (snapshot) of pipe flows and nodal pressure considering the known source head (water levels in the reservoirs) and known nodal demands. However, neither the nodal demands nor the water levels in the reservoir remains constant over the period of time. Nodal demand changes during the day and water levels changes due to filling and emptying of reservoirs. Therefore, analysis over extended period of time, say 24 to 72 hrs, is carried out to know variation of pipe flows, nodal heads and water levels in the reservoirs. This is called Dynamic Analysis (DA) or Extended Period Simulation (EPS). EPS is very useful in problems related with operating schedule of pumps and valves.

The solution approaches used to iteratively solve the set of non-linear equations are often controlled by several parameters. These parameters could be EPS run time step lengths or tolerance factors that signal the model when the solution has converged. The user must either specify the values for the solution parameters or accept the default values provided by software.

EPS describes the behaviour of the distribution when there is a variation of parameter, e.g., the demand changes in peak and non-peak hours. EPS analysis describes this correctly.

12.7 Design and Rehabilitation of Distribution System

The problem of designing of pipe WDS essentially involves determination of the location and the sizes of the different components that will meet the physical and operational requirements imposed on the network with minimum cost. The constraints include the hydraulic laws and operational ones such as minimum pipe sizes, restriction on commercially available sizes, and mainly the pressure requirements at critical nodes. As the system ages, the capacity of the system may not be sufficient to meet the growing demands. This may happen at the end of design period or even before that. Since the pipes have a lifespan longer that the usually adopted design period of 30 years, rehabilitation of pipelines are preferred.

12.7.1 Design of Water Distribution Systems (WDS)

The prime requirement in the design of a WDS is to minimise the cost, which is usually considered as an objective in optimal design problems. The total cost of the network is generally assumed to include the cost of the pipes, pumps, valves and other components, and present value of maintenance and operating costs. Several approaches have been suggested for minimum cost design as well as reliability-based minimum cost designs of WDS. Reliability in design assures systems performance under some abnormal conditions such as arising due to uncertainty in demands and pipe roughness values, failure of pumps, pipes and other components, or excessive demand condition such as fire demand. The cost of network increases with the increase in the level of reliability incorporated. Due to economic reasons, minimum cost designs giving satisfactory performance under normal working conditions are preferred.

The optimal design of a single source branched WDS is rather easy as flows in all the pipes can be fixed uniquely. Branched networks may be gravity-fed in which supply is from reservoirs or may be pumped one. Design methodology for such network is discussed in Chapter 6. The linear programming technique provides a global optimal solution and as mentioned earlier in Chapter 6, a cloud-based software “JalTantra” developed by IIT, Mumbai, can be used for design of single source branched WDS with limited number of pipes.

Loops are provided in WDS for better pressure management and to have an alternate path for supply of water to consumers. Usually, looped networks are the combination of branches and loops in which several branches emerge from loops. Traditionally, looped WDNs are designed by assuming various unknown parameters and checking the performance of the network to meet various design criteria using any methods of analysis. With the help of network solvers, several feasible designs are obtained and the one with minimum cost is selected. Designer can start with all minimum sizes, analyse the network, check available pressures at nodes and other criteria like pipe flow velocities. If all the criteria are found satisfactory, design is over. Else, some of the pipes having higher head loss gradient/higher velocities can be selected to increase the pressures. The process is repeated till a feasible design is obtained. The methodology is simple and straight forward, however, the labour and time involved in obtaining the design solution is entirely based on designer’s judgment and experience. Further, this approach has an element of doubt that a solution better in performance and lesser in cost than the selected one can be obtained.

Several optimisation methods based on differential calculus and mathematical programming techniques such as linear programming (LP), non-linear programming (NLP), and dynamic programming have been reported in the literature. The differential calculus and NLP-based approaches have a drawback in that they assume pipe diameter as continuous variable and at the end of optimisation, converts non-commercial size to commercial size. This conversion from continuous to commercial size makes the solution sub-optimal. On the other side, LP technique provides split pipe solution. The most promising linear programming gradient (LPG) technique is observed to terminate at a local optimal solution. Split pipe solution is not liked by many water authorities as: (i) it requires a reducer; (ii) one of the lengths of two sizes may be very small as compared to other. The dynamic programming-based techniques have the problem of curse of dimensionality in large practical-sized networks. Even though several algorithms were developed and tested for their efficacy on small networks, none of them were observed to handle large practical-sized networks with all its complexities. The optimisation techniques have advantages over traditional techniques that several feasible designs can be obtained by initiating the search from different starting points. The least costly feasible design can be selected. Software LOOP Vr. 4, developed by UNICEF, is freely available in public domain. It provides design based on the user-defined head loss gradient. However, this software is DOS-based. Some commercial software based on NLP techniques are also available.

In the last two decades, many evolutionary techniques that include genetic algorithm, simulated annealing, particle swarm optimisation, genetic evolution, cross entropy, Jaya algorithm, Rao-I and Rao-II algorithms, have been suggested for minimum cost design of WDSs. These algorithms search the entire solution space by starting search from several points and moving to the next generation either randomly or using some nature/bio-inspired techniques to improve the population. The search stops at the end of some pre-specified generation, and the best solution is considered as optimal one. Constraints are handled through penalty-based approaches. Hydraulic laws governing flows in looped WDSs are satisfied through network solver. As the search starts from several points, there are more chances of reaching to near global optimal solutions as compared to mathematical programming-based optimisation techniques. Also, several near optimal solutions are available at the end.

These evolutionary techniques consider some parameters that a designer has to decide. With the variation in these parameters different designs are obtained. Further, each run of the programme does not give the same solution. Therefore, several runs with different parameters are required which make the evolutionary techniques computationally extensive. The search efficiency can be increased by reduction in search space, self-adoptive penalty, and appropriate type of analysis to quantify constraint violations. Application of Critical Path Method (Bhave 1978) for search space reduction and penalty cost based on the capitalised energy cost to provide additional head to remove pressure deficiency in the network was suggested by Kadu et al. (2008) for improving genetic algorithm. However, these can be used for other evolutionary techniques. PDA instead of DDA was proposed by Abdy Sayyed et al. (2019) to obtain deficiency in available flows and available heads and use the same in obtaining combined flow-head deficit penalty. Even though these types of measures reduce the number of evolutions, the application of methodologies to large practical-sized networks require large number of evolutions and high-computational time.

Considering the advantages of mathematical programming techniques in quick identification, the local, if not the global optimal solution and global search capabilities of evolutionary techniques, hybrid algorithms by combining the two, have been proposed in the literature to reduce computational efforts. Also, hybrid algorithms by combining two different evolutionary techniques have been suggested. However, no software is freely available in public domain.

Methodologies have also been developed to include other objectives such as reliability, pressure equalisation, leakage reduction in a framework of multi-objective design problems.

Several optimisation methods based on differential calculus and mathematical programming techniques are discussed above, however, there are seldom available softwares which make use of these algorithms, moreover the softwares are costly and not user friendly. Some of the softwares have limitations of handling limited number of pipes. The practical method would be to employ the prudent parameters of velocity (generally around 1 m/s) and head-loss gradient (m/km), normally within 10 m/km. The optimization of pipe diameters can be iteratively achieved easily on any hydraulic model software as below.

12.7.2 Optimisation of Pipes in OZs

In most of the cases, some pipes in distribution system are existing and all OZs are to be reengineered with increased nodal demands, peaking factor, and revised norm of residual nodal head.

Prerequisite of Pipe Diameter Optimisation:

To begin with, complete the exercise of forming OZs and DMAs and making them hydraulically discrete as per **Annexure 12.2** should be carried out on hydraulic model. Every OZ should be served by a single service tank. There should be separate branch pipeline to each DMA from the main line (outlet) of the service tank (Figure 12.19). The maximum demand serving capacity of all the existing service tanks should be computed and their command areas, i.e., optimised boundary of operational zones be formed for existing tanks (Figure 12.11). This step is important to avoid the tank going empty or overflowing. In unserved areas plan new service tanks.

The pipes which have been decided to be discarded and replaced due to outlived or frequently leaking should be considered as new pipes in the hydraulic model. So also, new pipes shall be considered to increase coverage to 100%. The demand to all the nodes shall be assigned. For each OZ, the number of connections shall be measured using GIS and the boundary of the discrete DMAs in the OZ shall be decided in the hydraulic model. For each OZ, following steps are followed for optimisation of pipe diameters.

Principle Adopted for Iterations for getting Optimised Diameters

For reaching the best economical solution in minimum iterations, first visualise the general direction of OZ, i.e., in slopping direction. For increasing velocity more preference be given for decreasing diameters to pipes which are in transverse direction to the direction of slope. If desired pressure is not observed, more preference should be given for increase in diameter to pipes which are in the direction of slope.

Further Steps:

The steps to be followed are as under:

(a) GIS-based hydraulic model of the city/town under consideration is prepared. Since, the model is based on GIS, the pipe lengths between the nodes are automatically scaled out. Also, the pipes and junctions labels are automatically assigned by the network software in its corresponding tables. The default values of pipe material and its HWC-value shown in the table are changed to actuals. Lowest supply level (LSL) of ESR is assigned. The required diameters are assigned to each pipe, as per the experience and judgment of the designer.

(b) While preparing hydraulic model, the existing pipes are shown in colour code and the new pipes are added to increase the coverage to 100%. So, the model contains both the existing as well as the new proposed pipes.

(c) The elevations are assigned to each node using GIS-based contours and the demands are also assigned to all the nodes using the method of “Future population density method” as discussed in the advisory on “GIS Mapping of Water Supply and Sewerage Infrastructure” which is available at MoHUA website.

(d) In hydraulic model, initially a base scenario is created for the entire city’s distribution system. The data set in base scenario contains all the data fed above. We need to iteratively run the hydraulic model for each of the OZ. The logic is shown in Figure 12.3. Run the model. In each iteration, observe pipe diameters, velocity and hf (m/km) in the pipe table and nodal pressures in junction table.
(e) Select only new pipes in pipe table. Following steps are taken for new pipes keeping existing pipes undisturbed.
(f) Sort diameters in descending order, observe values of velocity and hf (m/km) in adjoining columns of the pipe table.
(g) Decrease diameters of the pipes above 100 mm in which velocities are too low and again run model.
(h) Observe the values of velocities in the pipe table. If velocity is less than 1 m/s and hf (m/km) is also less than 10 m/km, and minimum nodal pressure is also more than or equal to residual nodal head as per norm, the steps are repeated.
(i) The process is repeated for all the new pipes whose diameters are more than 100 mm, till we get all optimised diameters.

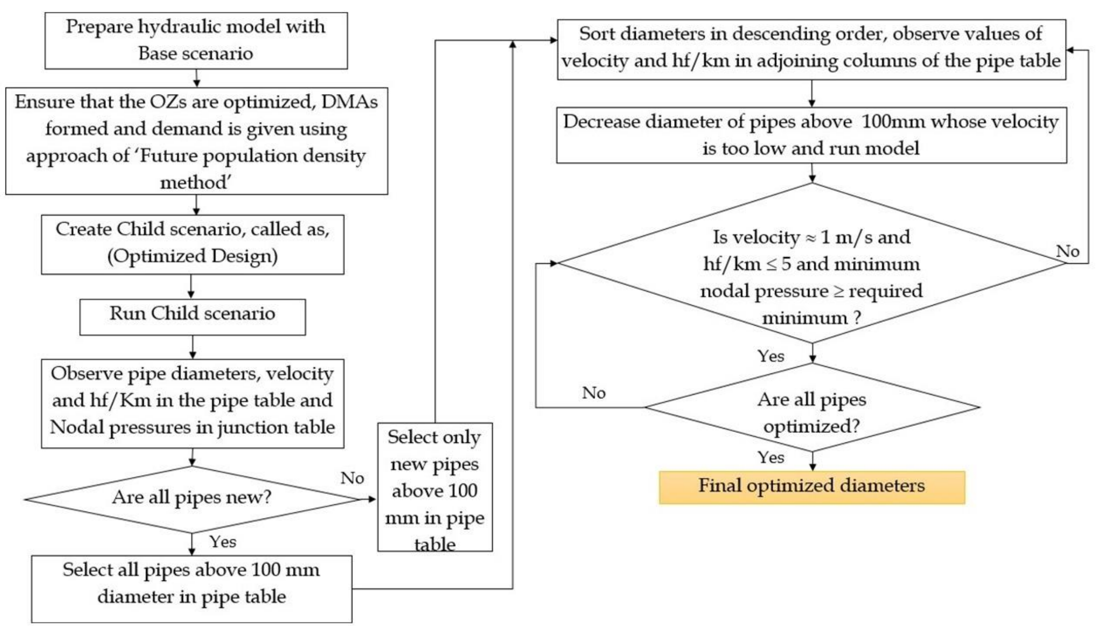


Figure 12.3: Iterative process of optimising diameters
(This iterative process is not performed by network software and is to be done by the designer)

(f) It can be easily seen that the diameters of existing pipes are retained and only the new pipes are optimised.
(g) It is experienced that one OZ can be optimised within half hour using this technique.

12.7.3 Rehabilitation of WDSs

The efficient and effective management of existing water distribution system (WDS) face challenges due to non-mapping of existing infrastructure, ageing of infrastructure, population growth, and extended urbanisation. Therefore, there is need for integrated solutions that support decision-makers to plan potential interventions considering the possible consequences and variations in mid- and long-term perceptions.

Modelling solutions, therefore, should be used to assess the rehabilitation of older WDS. The rehabilitation work may be necessary because of:

(a) the cumulative effect of tuberculation and scaling resulting in lowering of frictional coefficients such as Hazen-Williams C-value;

(b) increased demands due to new customers;

(c) excessive leakage;

(d) infrastructure improvements;

(e) water quality problems.

The problem associated with rehabilitation are somewhat more difficult than designing a new system. Problems include:

(a) working with existing piping;

(b) numerous conflicts with other buried utilities;

(c) the added importance of the condition of the paving;

(d) the large range of alternatives to be considered.

The one way in which rehabilitation analysis is simpler than other design application is that pressure zones and their boundaries are already defined and are usually not being adjusted.

Modelling Existing Conditions

The model of the existing system with known mapping of existing pipe network will reveal which pipe segments have bottlenecks. These will usually be the segments with higher velocities or higher hydraulic gradients. Field data should then be collected to corroborate the simulation results. Those segments that have bottlenecks will need to be replaced, paralleled, or rehabilitated. In general, peak hour demands and fire flow demands are the controlling conditions, and steady state runs may be used to solve this problem.

Usually, replacing the piping is the most expensive alternative and should not be selected unless the existing piping is in poor structural condition. The decision of whether to parallel or rehabilitate the existing piping depends upon the design flow in the area. Rehabilitating the existing pipes will increase the nominal diameter of the pipe. If the future flows are going to be significantly greater than the original flows in the pipes, then rehabilitation will provide sufficient capacity, and new pipes roughly paralleling the old system are needed.

The retrofitting of DMA, OZ, and pressure zone is incorporated in **Annexure 12.2**.

If the existing intermittent water supply system with existing ESRs of a project area is to be converted into a 24×7 continuous water supply system, in which various scenarios may come across with retrofitting and rehabilitation of distribution system, in which the required residual heads are not achieved, then the corrective measures need to be taken. The measures to increase the residual nodal pressures have been illustrated with different scenarios in **Annexure 12.3**

12.8 House Service Connections

12.8.1 General

The supply from the street main to the individual buildings is made through a house service connection. This consists of two parts, viz., the communication pipe which runs from the street main to the boundary of the premises, and the service pipe which runs inside the premises. The communication pipe is usually laid and maintained by the local authority at the cost of the owner of the premises while the service pipe is usually laid by the consumer at his cost.

The service connection including the details of the internal plumbing system should conform generally to the National Building Code and particularly to the bye-laws of the concerned local authority. Extreme care should be bestowed for the design and construction of plumbing system. The rational design criteria evolved by CBRI for plumbing should be followed.

12.8.2 System of Supply

The water supply in a building may be through one of the following or combinations of both depending upon the intensity of pressure obtained in the street main and the hours of supply.

(a) Direct supply system, and

(b) Downtake supply system with or without sump and pump.

If the pressures near the premises are adequate to supply water for 24 hours to the water fittings at the highest part of the building, then suitable connections may be allowed to deliver water directly bypassing the consumer's underground storage tank. In cases where the pressures in the street mains are not sufficient to deliver water supply directly, the downtake supply system with ground level storage and boosting is adopted. Direct supply system is recommended under one condition only when the number of floors in a building is not more than three.

One or more number of connections of appropriate sizes of entry pipe (house service connection pipe) may be granted to any multi-storey building and group of such buildings depending on number of households in the premises. However, with proper diameter of entry pipe, no. of connections should be as less as possible. Accordingly, the ULB can make suitable changes in their bye-laws.

The supply in any case is controlled usually by a ferrule (IS 2692:1989 R2003 "Ferrules for Water Services" may be referred) (Ferrules are commonly used in taking out branch lines from water mains and also in stopping supply to branch lines where so desired). The ferrule is a draw-off appliance with a vertical inlet for screwing on to water main and a horizontal outlet and closed by means of a washer plate carrying a renewable washer which shuts against the water pressure on a seating at right angles to the axis of the threaded plug which operates it. The tapping of the street main should never be on the side or bottom of the main, which is throttled sufficiently to deliver the required supply at the minimum residual pressure of 17-21 m or 12-15 m, as the case may be. The supply is also controlled by a stopcock at the beginning of the service pipe. A meter is to be installed beyond the stopcock for measuring the flow (details are provided in Chapter 13 of part A of this manual). Any temporary disconnection of the supply is made by the stopcock and any permanent disconnection is made at the ferrule. The size of the ferrule should not exceed a quarter of the nominal diameter of the municipal service main and also be less than the size of the communication pipe. If a larger size of connection is required, a branch with the required number of common service pipe can be used. Where the pipe has to cross a drain, a suitable encasing pipe may be provided for prevention of cross-connection.

12.8.3 Downtake Supply System

Details are enclosed in **Annexure 12.6**.

12.8.4 Materials for House Service Connection

The various pipes used for service connections should conform to the relevant Indian standards.

a) Normally G.I. pipes are used for service connections. They have the advantage of low cost and high strength. They suffer from the disadvantage of shorter lifespan in corrosive soils especially at the screwed joints. Bituminous covering for the pipe increases its longevity. The carrying capacity of the pipe may also be reduced due to incrustation. Rigid PVC pipes, medium density polyethylene (MDPE - ISO 4427) and composite material polyethylene-aluminium-polyethylene (PE-AL-PE) pipe (IS 15450: 2004) are also coming into use. These pipes are flexible and light, and the carrying capacity is not reduced with age due to incrustation. Presently, MDPE pipe is widely used for house service connection due to its low cost as well as having other advantages. They, however, are liable to be damaged easily. They also soften at temperatures above 65 °C and as such, cannot be used in hot water system.

b) The communication pipe is attached to ferrules or saddles depending on the material of the distribution main in the street. Gun metal or bronze ferrules are screwed into the street main while special screwed saddles are fixed on cement asbestos and PVC pipes.

c) To measure the quantity of water used by the consumers usually suitable size of domestic water meters are fixed on the service pipe immediately after the stopcock in the consumer premises in a masonry pit.

12.8.5 Meters and Metering of House Service Connections

The nominal sizes of domestic water meters are varying from 15 mm to 50 mm as per {IS 779: 1994 (Reaffirmed 2015)} and bulk water meter is varying between 50 mm and above as per {IS 2373: 1981 (Reaffirmed 2017)}. Sizing of water meter is done keeping in view the guidelines given in Indian standard {IS 2401: 1973} and {ISO 4064 Part-II: 2014}.

In general, main considerations are as follows:

i. Water meter should be selected according to the flow to be measured and not necessarily to suit a certain size of water main.
ii. The maximum flow should not exceed the maximum flow rating.
iii. The nominal flow should not be greater than the nominal flow rating.
iv. The minimum flow measured should be within the minimum starting flow of the meter.
v. Low head loss, long operating flow range, less bulky and robust meter should be preferred.

Automatic meter reading water meter/Smart digital water meter are also used widely as they have various advantages over conventional water meters for domestic purposes.

12.9 Protection Against Pollution Near Sewers and Drains

12.9.1 Horizontal Separation

A water main should be laid with at least three metres of horizontal separation from an existing or prospective drain or sewage line. If the three metres horizontal separation is not achievable, then the water pipeline should be internally lined and properly encased within outer M.S. pipe while laying it. Such vulnerable spots should be visited by the competent authority from time to time to monitor any leakages.

12.9.2 Vertical Separation

When water mains cross a lateral sewer line, storm drain, or sanitary sewer, they should be installed such that the bottom of the water main is 0.5 m above the top of the drain or sewer, and the mains are as far away from the sewer as feasible. This vertical separation should be maintained for a distance of 3 m on both sides, measured normal (perpendicular) to the sewer or drain it crosses.

12.9.3 Unusual Conditions

When conditions prevent the minimum vertical separation set forth above (minimum from being maintained), or when the water main must pass through a sewer or drain, the water main should be laid with double flanged DI pipe with rubber gasket joints for a length on either side of the crossing. When creating such crossings, it is better to use double flanged DI pipe with robber gasket joints for the sewer, and to have both the water and sewer mains pressure tested before backfilling.

When a water main has already been installed and a new sewer is being installed, the above factors may be considered, and the water main may be realigned to the extent necessary if the sewer cannot be installed in accordance with the above recommendations.

Where the water mains are crossing the sewer mains, such vulnerable spot shall be shown on GIS maps as well as asset management maps.

12.9.4 Protection Against Freezing

Details of protective measures are described in Chapter 7 “Distribution System” in Part B of the manual. Non-metallic pipes are discouraged in such environment.

12.10 Water Distribution Network Model

Water distribution network models have been widely accepted within water utility industry as a mechanism for designing, analysing, and simulating hydraulic behaviour in the networks by generating different scenarios. For Drink from Tap (24×7) water supply schemes, hydraulic modelling is necessary. In earlier times, when computers were not invented, this technology of hydraulic modelling was not available and therefore the distribution systems were designed using simple Excel sheet methods and were based on the rule of thumb and personal experience of engineer. The perception of distribution network was vague. Therefore, the bigger networks could not be designed rationally, and this is one of the reasons of resorting to intermittent water supply.

However, powerful softwares are now available, which are in use for designing the distribution pipe network. With correct data fed to the models, they give fast and accurate results of analysis and design. By modelling a system, we gain a full understanding of hydraulic behaviour of the system. Hydraulic model is used as a tool to plan infrastructure improvements, develop operation and maintenance (O&M) strategies, and proactively manage water system.

Current water distribution modelling software are capable of generating model of any city. Many software packages are integrated with GIS and CAD technology to facilitate model construction, storage, and display model results. Modelling capability was expanded to include EPS that could accommodate time varying demand and operations with controlling elements such as check valves, flow control valves, pressure regulating valves, fire hydrants. The modelling can do analysis of pressure deficient water networks as well.

12.10.1 Inside Working of Hydraulic Model

Hydraulic model is defined as the process of creating a representation of network modelling of actual water supply or sewerage system using computer software. It is a mathematical model of fluid dynamic. Network flow involves two basic principles - conservation of mass and conservation of energy. The principle of conservation of mass involves the continuity equations which means at any node, the flow coming in must be equal to the flow going out. Principle of conservation of energy is used in forming energy equations in which frictional head loss between the two nodes is computed which is then used to compute hydraulic grade level (HGL) of the downstream side node. Continuity equations are of linear nature whereas energy equations are non-linear. Therefore, the convergence methods such as Hardy Cross, Linear Theory, Newton-Raphson, and Global Gradient are evolved. Among them, the Global Gradient method is widely used in the computation engine of all the software.

12.10.2 Establishing Objectives

Prior to applying the model, the specific modelling objectives should be clearly established. The objectives may include specification of water demand and operational modes. Based on these, scenarios can be defined, and the model is applied appropriately.

12.10.3 General Criteria for Selection of Model and Application

The basic scope and needs of the modelling process should be initially defined to select an appropriate software package to satisfy not only the specific needs of the current project but also likely future needs. Factors that may be used in the selection of a software package include:

(a) Technical features
(b) Training/support and manuals
(c) User interface
(d) Integration with other software (such as GIS, CAD)
(e) Cost
(f) Response from users
(g) Capability of analysing the network in draught conditions
(h) Scenario Management

12.10.4 EPANET Freeware Software

EPANET 2.0 is software application used round the world to model WDS. Engineers use EPANET 2.0 to plan and size new water infrastructure, upgrade ageing infrastructure, water quality simulation and prepare for disasters. It can also be used to simulate contamination hazards and assess resilience in the face of security threats and natural disasters.

EPANET 2.0 is a Windows-based software that is in the public domain. This may simulate the hydraulic behaviour of a pressurised pipe network, which includes pipes, valves, storage tanks, and reservoirs, over a long period of time. It can be used to monitor the flow of water via each pipe, the pressure at each node, and the water level in each tank, among other things. As mentioned earlier, the recent version of EPANET, EPANET 2.2, is capable of pressure-dependent analysis. This software can be integrated with other software developed in C++, MATLAB for various other uses like design, optimal location of various facilities like booster pumps, booster chlorination, online optimal sensor locations for pressure monitoring, identifying leakages, and contamination.

Above all, EPANET (hereafter EPANET, is used for both the versions of EPANET) assists water companies in maintaining and improving the quality of water distributed to consumers.

12.10.5 Developing a Basic Network Model

The basic network model should first be characterised. The model should be developed based on accurate up to date information. Information should be entered carefully and checked frequently. Following the entry of the data an initial run of the model should be made to check for reasonableness.

In hydraulic model, the distribution system is represented as series of links and nodes. Links represent pipes whereas nodes represent junctions or junctions where change in diameters occurs, sources tanks, and reservoirs. Valves and pumps are represented as either nodes or links depending on specific software package. Building a network model, particularly if a large number of pipes are involved, is a complex process. The following categories of information are needed to construct a hydraulic model:

(a) Characteristics of pipe network components (pipes, pumps, tanks, valves)

(b) Water use (demands) assigned to nodes (temporal variation required in EPS)

(c) Topographic information (elevation assigned to nodes)

(d) Control information that describes how the system is operated (e.g., mode of pump operation)

(e) Solution parameters (e.g., time step, tolerances as required by solution techniques)

It is required to create a model comprising of all these components. Initially, a network of pipes and junctions is created using “Model Creator” facility of the hydraulic network software. Once the existing and new pipes are added, the shape files of the pipes are used in the process of building the model.

12.10.6 Network Inputs

Identifying pipes to include in the model is the first step in building a network model. Nodes are typically located at pipe junctions or key facilities such as (tanks, pumps, control valves) or when pipe parameters such as diameter, 'C' value, or construction material change. Nodes can also be put at

known pressure locations, sampling sites, or places where water is consumed (demand nodes). The required pipe network component information includes the following:

(a) Pipe diameter, length, and roughness factor

(b) Pumps (pump curves)

(c) Valve (settings)

(d) Tank (cross section information, minimum and maximum water levels)

Construction of pipe networks and its characteristic may be done manually or through use of existing spatial data bases stored in GIS or CAD packages.

12.10.7 Integration of Model with GIS

A Geographical Information System (GIS) is a powerful configuration of computer hardware and software used for compiling, storing, managing, manipulating, analysing, and mapping spatially referenced information. It integrates database operation with visual and geographic analysis functions enabled by spatial data. GIS can serve as an integral part of any project that requires management of large volumes of digital data and the application of special analytical tool.

Model/GIS integration is a three-step process:

i. Interchange: Data are exchanged through an intermediate file which may be an ASCII text file or spread sheet. Data is written to this intermediary file, where it is reformatted for the model if necessary and then read into the model. The model and GIS run independently.

ii. Interface: Links are built between the model and GIS. These links are used to synchronise the model and GIS. The data are duplicated on each side of the link and the model and GIS are run independently. One common approach is the use of shape files which can pass data between the model and GIS and optionally update either based on the data contained in the other.

If the data of existing pipeline is not available and the scheme needs to be designed afresh then pipe and junction can be modelled using road centre lines of the city. The road centre line can be created either by digitising the road or using the freeware such as Open Street Map.

iii. Integration: A single repository for the data is used. The model can be run from the network software or from the GIS.

Hydraulic Model/GIS integration leads to the following benefits:

1. Time saving in constructing models.
2. Accurately and quickly feeding the data to the hydraulic model.
3. For very big networks such as for Class 1 and Class 2 cities, manually assigning data is prone to the error and very difficult. In this case GIS integration helps.
4. Ability to integrate land use, demographic, and monitoring data using GIS analysis tools to predict future system demands more accurately.
5. Visual map-based quality control of model inputs.
6. Map-based display and analysis of model outputs in combination of other GIS layers.

Integration of network software with GIS is shown in Figure 12.4. Resulting hydraulic model uses combined capabilities of both software.

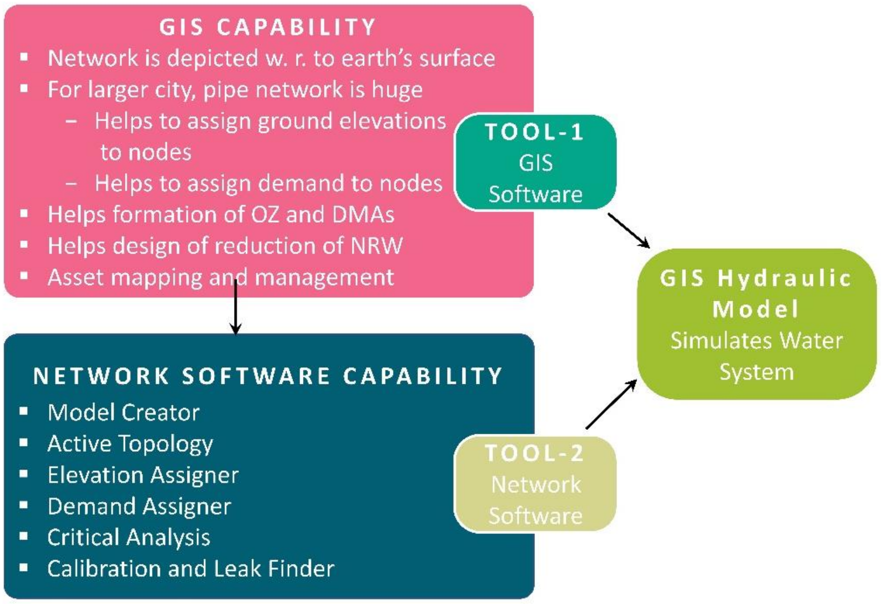


Figure 12.4: Integration with GIS

12.10.8 Creating Hydraulic Model using Network software

The first step for preparation of hydraulic model is creation of the base maps.

Base Map: Preparing hydraulic model of a city needs base maps. Base map consists of (i) Satellite image (ii) Digitisation of water supply components (iii) Landmarks (iv) Existing water infrastructure and (v) Contours.

Creation of base maps is discussed in Section 2.8 of Chapter 2 of Part A manual.

GIS-based map of the city, showing footprint of buildings, land use areas like residential area, commercial area, parks, gardens, and roads, is added as background layer in hydraulic network software. In base map, existing water infrastructure comprising of existing tanks, existing pipelines are added by the respective shape files.

Existing Pipelines: For creation of hydraulic model, we need the maps of existing pipelines, and all other relevant data such as existing valves and their status whether they can be used further. Two cases may crop up: (a) data of pipelines is available; and (b) data is not available.

(a) Data of existing pipelines available: Some Urban Local Bodies (ULBs) maintain their network data satisfactorily, but it is available in AutoCAD format. In such cases, the pipelines along its attributes are geo-referenced. Geo-referencing is the process of assigning real world coordinates to each pixel of the raster image. After geo-referencing, the shape files of the pipelines are created. If the data is available only in the hard copy format, then the hard copy of the map showing the pipelines is scanned and its JPEG (Joint Photographic Experts Group) file is created. The JPEG file is added to the GIS software, geo-referenced and then converted to the shape files. The data of pipe attributes in this case may be assigned manually.

(b) Data of existing pipelines not available: If maps of alignment of existing laid pipelines are not available, then the task becomes difficult. In such situation, data of existing laid pipelines is obtained by conducting pipeline alignment survey. The survey team should comprise of the ULB's engineer, meter readers, valve operators and contractor's staff. Using the Global Positioning System (GPS) instrument, the alignment should be marked on GIS map. In this case, the team should visit the site of pipe alignment and interact with the customers residing in the area. After discussion with them, alignment of pipes is identified. The trial pits should be taken at suitable intervals so that the team can understand and note the attributes of actual pipes laid. These attributes such as pipe material, diameter, and the year of laying of pipelines are then marked. In this way the existing pipes and valves are marked on the GIS maps. Recently, IIT Chennai invented some indigenous technology of condition assessment of pipes which is described in Chapter 2 (Planning) which can be used.

Combining Existing and New Pipes: New pipes are added in the area where they are required for making 100% coverage. Care should be taken to add new pipes only in the areas where they are needed. For example, there can be reserved areas like cantonment areas, industrial area, etc., wherein respective authorities may have their own water supply system. In such areas, pipes need not be shown in the hydraulic model. Once the shape files of the existing and new pipes are available, they are combined.

Data: Data to be given are (a) levels to reservoir, tank, and all the junctions; (b) demands to the junctions; (c) pipe attributes like diameter, material, C-value, etc. Each demand node (tank) supply water to the respective OZ. Hence demand of OZ is assigned to such demand nodes. If the model is to be prepared using GIS, the data of lengths of pipelines need not be given as they are automatically scaled out, however the data can be given manually too. Most important job is to assign levels and allocate the demands to the hundreds of junctions in distribution network. To account for 10% of minor losses, length for pipes can be increased by 10%, or nodal demand can be increased by 5.28%, or C-value can be reduced by approximately 5%.

Creation of Active Topology: Using shape file of pipes and employing the Model Creator tool of the network software, the active topology of the existing and new proposed pipelines is created. New pipes are proposed to make 100% coverage.

Assigning Data to Nodes: Manually assigning data to any huge network is extremely difficult and it is prone to errors and moreover, may not be accurate. GIS helps to solve this problem by integrating it with the hydraulic network software (Figure 12.4). Using GIS, values of ground elevations and demand of water are given to each node of distribution system.

Assigning Ground Elevations to Nodes: Assigning ground elevations to the nodes is described in Figure 12.5. The Elevation Assigner tool of hydraulic network software assigns ground elevations accurately and in quick time to each node. The computation is based on the nearest value of elevation from the GIS contours. Only condition is that the GIS contour map must exactly sits over the georeferenced base map that contains the layer of pipe nodes. This requires the same co-ordinate system for both the layers of contours and pipe nodes.

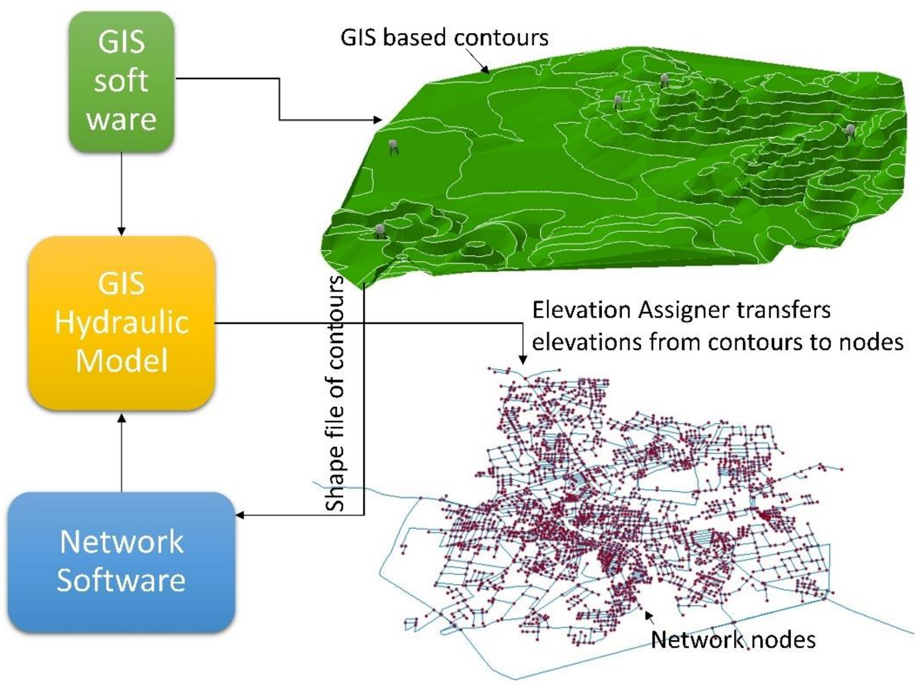


Figure 12.5: Assigning elevations to nodes using network software

12.10.9 Water Demand Inputs

Water demand is the driving force behind the operation of a WDS. The water demands are aggregated and assigned to nodes. It is important to be able to determine the amount of water being used, where it is being used and how this usage varies with time. An adjustment factor should be used to account for losses and other unaccounted water usage so that total usage in the model corresponds to total production.

Assigning Demand to Nodes: The Demand Assigner tool of the hydraulic network software assigns values of demands (Load) to nodes. The computation is based on the 'Thiessen polygon' and the 'Future Population Density' layer (as detailed in **Annexure 2.7** in Part A of this manual). A Thiessen polygon is generated around each node of the pipe network.

Hydraulic network software generates polygons by a series of perpendicular bisectors of a line joining the two adjacent nodes (Figure 12.6), and then forming polygons around each node (Figure 12.7).

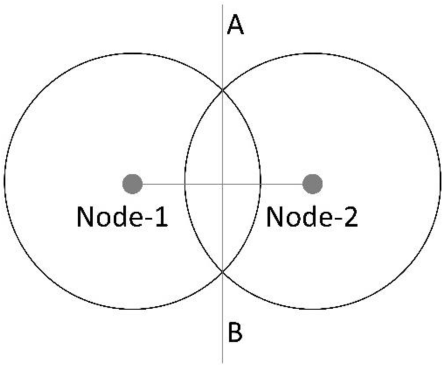


Figure 12.6: Perpendicular bisector of a line joining the two adjacent nodes

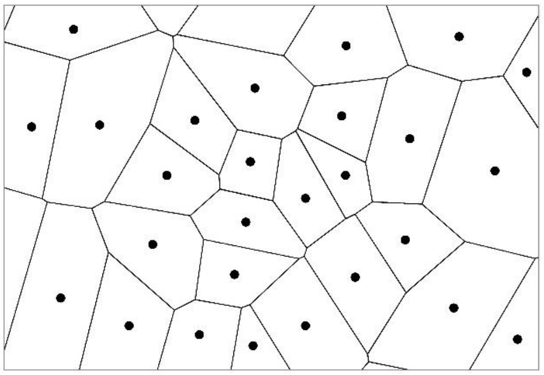


Figure 12.7: Polygons formed around each node

Second important task is to assign the demands to each node. It is carried out by the tool called Demand Assigner (Figure 12.8). It interpolates the population density in each polygon of the Thiessen polygons and then assigns value (depending upon the land use) to the node inside that polygon.

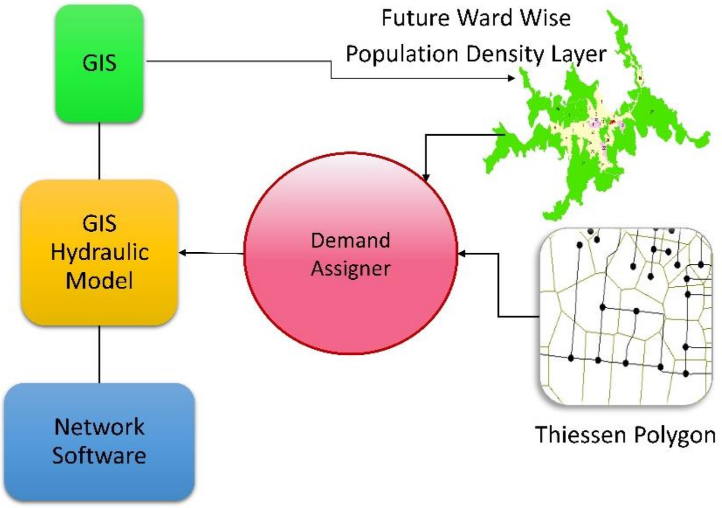


Figure 12.8: Assigning demand to nodes

There are three methods for processing demand data, these are: point load, area load, and population/land use data. If the consumer data generated through survey is to be used, then the method would be point load.

Assigning Elevations and Demand to Nodes: Using GIS, values of ground elevations are assigned to each node of distribution system by using facilities of Elevation Assigner and Demand Assigner available in the network software.

New Reservoirs: Boundaries of the OZs of the existing service reservoirs should be decided utilising the logic as discussed in the subsequent paras. However, after marking the optimised boundaries of the existing reservoirs, there still remains some of the areas that are unserved by any of the existing tanks. In such unserved areas, new service tanks should be proposed.

12.11 Operational Zones

The Water Distribution System (WDS) of a city may consist of several reservoirs (elevated/ground), floating reservoirs, pump stations feeding to the network. Location of leaks through instrumental methods is a challenging task, especially when the size of leak is small. It is therefore necessary that network of the city is divided into smaller parts for ease in operation. OZs can be formed for each individual reservoir depending upon the situations of reservoirs. OZs can be further divided in smaller area called district metered area (DMA) that isolate small group of nodes for feeding most preferably from a single inlet pipe provided with water meter.

OZ is the jurisdiction of each tank to serve water supply. Performance of distribution of water depends on size of OZ of tank. A schematic of the operation zone with DMAs is shown in Figure 12.9.

12.11.1 Design Criteria for OZs

For the approach of 24×7 system, following are design criteria for OZs to supply water equitably and with required pressure:

1. Compute optimum demand that a tank can serve and based on that, extent (boundary) of an OZ should be determined so that when in full operation, the tank should not get empty, or will overflow.
2. The minimum nodal pressures are fulfilled.

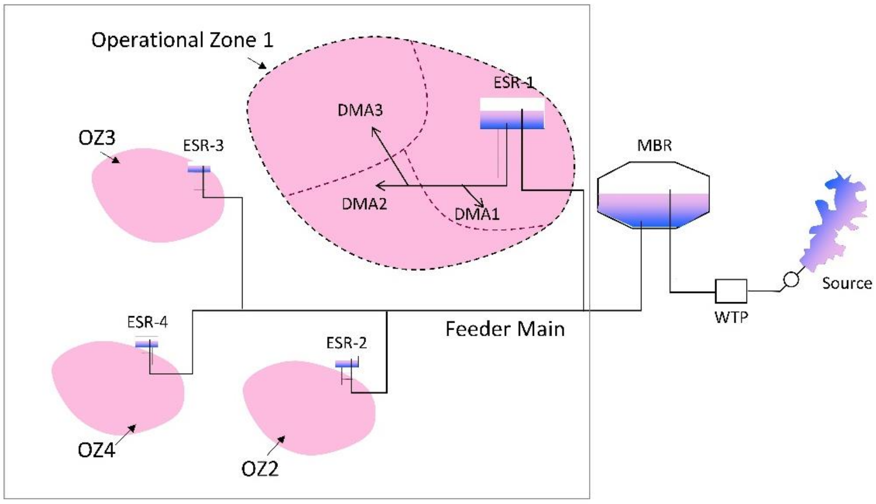


Figure 12.9: Operational zone with DMAs

12.11.2 Developing OZ on Hydraulic Model

A hydraulic model should be created as described in above Section 12.10. With base map, the pipe network, including mapping of the existing and new service reservoirs, is modelled on the hydraulic model. It is required to determine the optimum boundaries of each of the service tanks. Initially determining the optimum boundary of existing serve reservoirs is discussed for which the optimum demand of the existing service reservoirs is required.

Computation of optimum demand that an existing tank can Serve: Optimum demand for an existing tank can be computed by mass balance curve method. In any case, it shall not exceed three times the volume of tank. Knowing the diameter of existing tank and water depth volume of the tank can be computed.

12.11.3 Fixing Optimum Boundary of OZ

To begin with, it has to be borne in mind that if the excessive capacity of the existing ESR remains unutilised, then increase the spread of OZ. Try to add adjoining area with lower elevations. Finally arrive at optimum boundary of OZ and the optimum demand that can be served by the existing tank. Steps involved are:

a) Commanding elevation of ESR = LSL of ESR which is equal to ground level + Minimum residual nodal head + 5 m for head loss in OZ. Below this commanding elevation, all nodes will approximately receive water with required head. The value of 5 m can be lowered or increased by the designer with his experience/prudence considering location and slope in OZ.
b) With an intention to use the optimised capacity of existing ESR, decide the boundary of OZ considering natural boundaries like road edges, stream, railway line, etc.
c) Find out the total demand of the nodes in the chosen boundary of OZ. It should not be more than the demand calculated as mentioned in para. no 12.11.2. If the total of demand of the selected nodes is much less than the optimum demand, that means capacity of ESR is underutilised and expansion of the boundary of OZ is required.
d) Above iterative process should be carried out by the designer, then the optimum boundary of OZ and optimum demand that an ESR can serve is computed.

The details are explained with Figure 12.10.

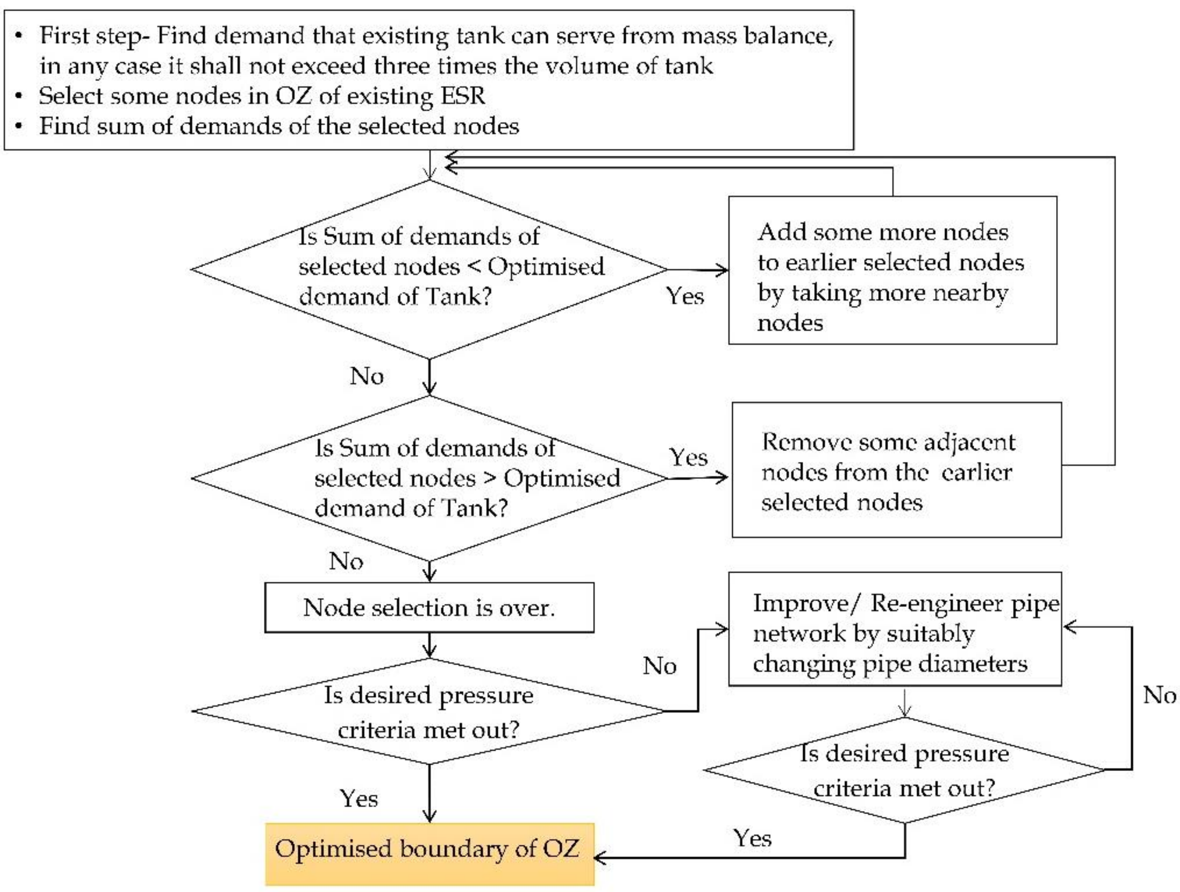


Figure 12.10: Logic for fixing boundary of OZ of existing tank (K.S. Bhole, 2015)

A hydraulic model should be created using the existing pipe as well as the new pipes. Suppose the network is as shown in Figure 12.11 (a). Objective is to decide and fix the extent of the OZ of this existing tank. Process of selection of nodes to fix boundary of OZ of existing tank is shown in Figure 12.11.

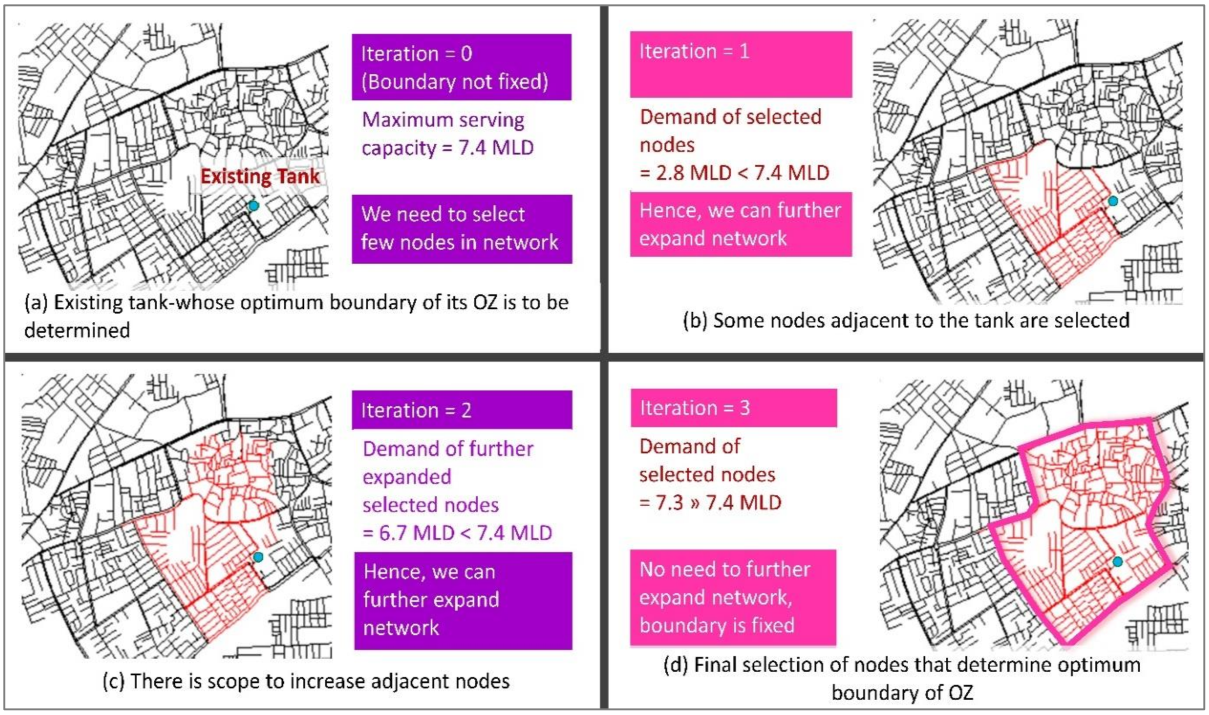


Figure 12.11: Process of selection of nodes to fix boundary of OZ of existing tank

The process starts with iteration 0. Let's assume the optimum serving demand of the existing tank is 7.4 MLD. (Figure 12.11 (a)). In iteration 1, some nodes in the vicinity of the existing tank are selected as shown in red colour in Figure 12.11 (b). The sum of demands of these selected nodes is 2.8 MLD which is less than optimum demand that the tank can serve. Hence, there is scope to increase the nodes and hence expand the boundary of OZ. In the third iteration some more nodes in the adjoining are of already selected nodes are added and the sum of such added nodes is computed. It is observed that the sum of demands of all the so far selected nodes is 6.7, hence, there is still scope of increasing the number of nodes. In the fourth and final iteration, few more nodes are added further and the sum of all the selected nodes is computed which comes out to be 7.3 MLD. Thus, in the fourth iteration, the demand of the selected nodes is approximately equal to the optimum demand of the existing tank. It shall then be checked for desired pressure criteria as mentioned in Figure 12.10. In this way, the optimum boundary of the existing tank is determined.

After fixing boundaries of all the existing tanks, there may be some nodes of the network which are unserved by all the existing tanks. A new tank should be suggested to take care of the unserved area.

If the OZs are not hydraulically discrete, any connecting pipe between the two OZs should be provided with isolation valves which will remain in closed condition and can be opened in case of any emergency condition to transfer water from one OZ to other.

12.11.4 Optimisation of Pipe Diameters

Major portion of the capital cost of the project is that of the cost of pipes which is about 70%. Therefore, many researchers have developed and studied different optimisation techniques to optimise the cost of pipelines ensuring that various hydraulic design constraints like pressure, velocity and head loss gradient are satisfied. The mathematical algorithms either run on an independent programme or on the top of hydraulic models. Optimisation techniques that have been studied include methods such Genetic Algorithms, Linear Programming, Non-Linear Programming, etc.

Some of the network software have a built-in facility of optimiser tools. But that software is costlier. Many designers face difficulty in understanding and using this tool while designing their projects for pipeline optimisation. They often complain that the tool results in non-telescopic pipe diameters that faces rejection from designers and the utility engineers. Besides this, some of the software do not render true and fully optimised results as they fail to observe the basic criteria of minimum velocity, say 0.4 to 0.6 m/s and few of the pipes in the optimised output of these software depicts very low velocity of even 0.1 m/s or so for the pipe diameter above minimum diameter, eventually these pipes are having unduly large diameter.

An illustrative example of Optimisation of diameters is carried out for one operational zone of Ahmedabad city which is enclosed in **Annexure 12.7**.

Design Principles for Optimising Diameters

While designing network for OZ/DMA, designer assigns the required data of LSL of water in ESR, pipe data such as diameter, material and C-value and the junction data like the ground elevation, nodal demands, etc. The parameter of giving LSL of water of ESR is based on following:

1) Achieving higher velocity reduces the diameters which reduces capital cost.
2) Reduced diameters mean less volume of water in the network within OZ and DMA, it takes less time to build up pressures after starting water supply on every cycle of suppling water.
3) Reduced diameters mean, easy, less time taking and less cost for repairs/replacement of pipes.
4) Provide appropriate staging height of ESR to achieve above.
5) Even though it is ideal to provide ESR at higher elevation and at centre of OZ fulfilling both conditions, this type of arrangement is seldom possible. Try to fulfil them to the extent possible.
6) LSL of water of ESR should be equal to highest ground level in OZ + minimum residual head + head loss for getting desired velocity.
7) Velocity should not be less than 0.3 m/s in all diameters above 80/100 mm.
8) The minimum diameter in the distribution system of city/town should be 100 mm for class I cities, and for others, 80 mm.

12.12 District Metered Area (DMA)

UK Water Authorities Association in 1980 introduced the concept of DMA. They used DMA management technology for monitoring leakages in water distribution networks (WDNs). The WDS before and after advent of DMA is shown in Table 12.1.

Table 12.1: WDS before and after advent of DMAs

SN	Before DMAs (1980)	After DMAs
1	Limited Flow Measurement	Flow and Pressure - measured at DMA inlets
2	Insufficient knowledge of distribution network	Improved Leakage Control
3	Limited ability to prioritise	Priority is known
4	Leakage control was passive	Leakage control is active
5	Low working morale	Working with confidence

When compared with the centralised system, DMA management has been reported to have several advantages such as better control over the system that resulted into reduction of water losses and helped pressure control. Thus, forming DMAs is an important task that helps in removing unreported leaks in a distribution network. The process is called active leakage control. DMA is therefore the building block of 24×7 continuous water supply.

A DMA is a sub-zone within OZ of a water distribution network that can be hydraulically isolated and for which water consumption is measured using water meters. Bulk flow meters are installed at the entry points of the DMAs, and all user connections are properly metered for recording the consumption.

If the network is separated into smaller sections, the flow, pressure, and control of the NRW can be better handled. The main purpose of DMA is to identify and prioritise leak identification and repair programme by computing NRW values. Another important purpose of DMA is to rationally distribute the water according to the needs with equal pressure.

A typical DMA scheme is shown in Figure 12.12 and a single typical hydraulically discrete DMA is shown in Figure 12.13.

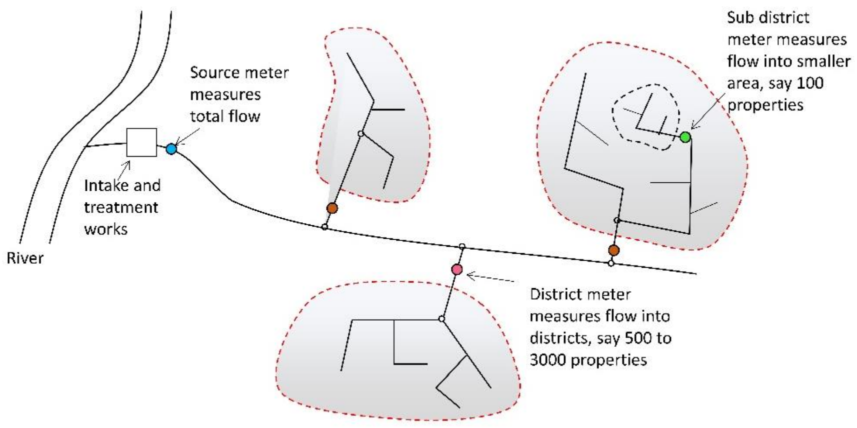


Figure 12.12: Typical DMA Structure

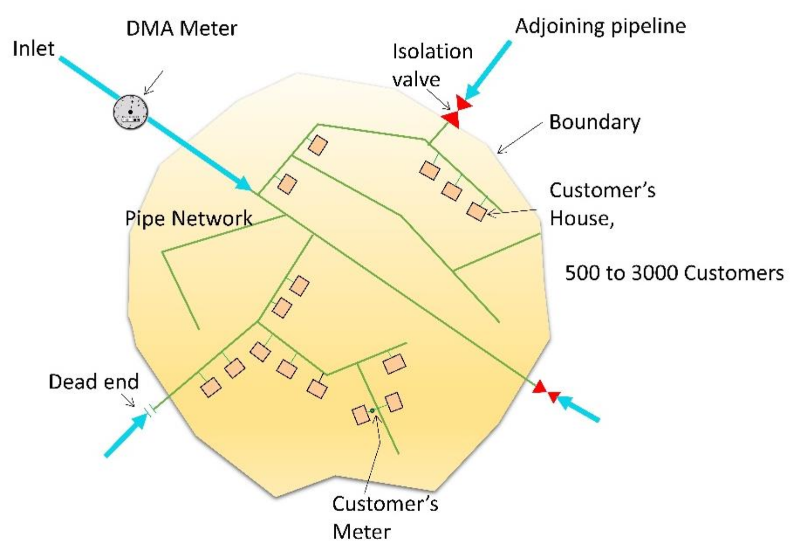


Figure 12.13: Typical hydraulically discrete single DMA

(A) TYPES OF DMAs

DMAs are categorised into four types: (a) Single inlet DMAs, (b) Multiple inlets DMAs, (c) Cascading DMAs, and (d) Pressure Managed DMAs. The four types of DMAs are shown in Figure 12.14.

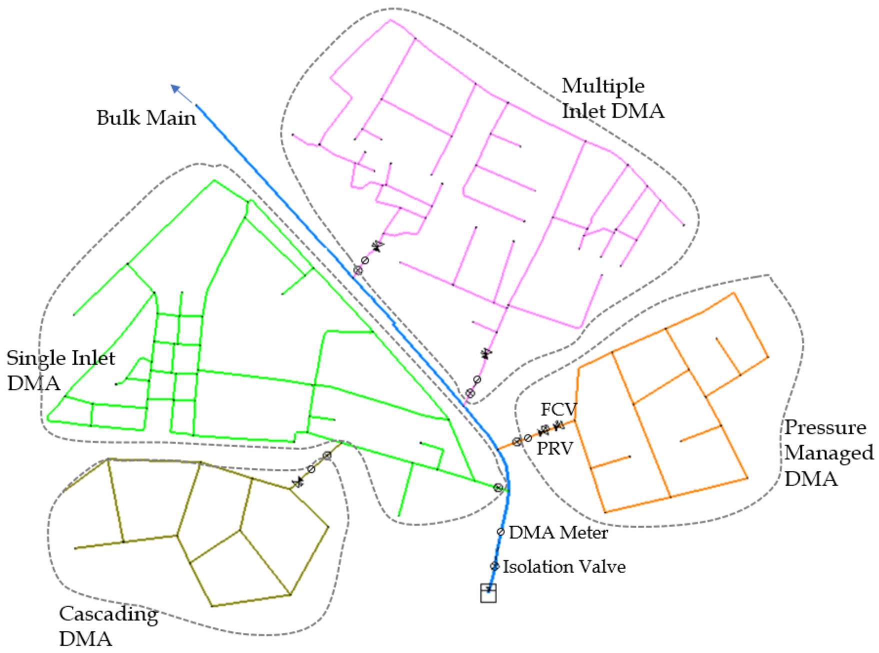


Figure 12.14: Types of DMA

(a) Single inlet DMAs: In this DMA, there is only one inlet meter, hence it has minimum errors.
(b) Multiple inlets DMAs: In the situations like pressure or system redundancy one meter is not possible and multiple inlets are to be installed. This arrangement is suitable when the width of DMA is less, and the length is more and hence, the pressure drop along the length is excessive. However, this involves extra cost due to providing of extra inlets with necessary isolation valve, bulk meter, PRV, and FCV.
(c) Cascading DMAs: Due to topology of the network, sometimes a DMA is fed through bulk meter of other DMA.
(d) Pressure managed DMAs: If the DMA is situated at low elevation, PRV is introduced to dissipate excess hydraulic pressure. It may be noted that PRV is not an essential feature and is not required to be provided for DMA on flat terrain.

12.12.1 Design of DMAs

Dividing the hydraulic systems into districts represents a technically structured approach for several management purposes including monitoring, control, and operations that might require the isolation of some portion of the system (e.g., in case of planned or accidental interruptions).

The sub-zoning of the distribution network should incorporate a DMA approach.

Criteria for initial DMA design are:

- Size of DMA - number of connections
- Number of boundary valves - must be closed to isolate the DMA
- Number of flow meters - to measure inflows and outflows in case of cascading DMA

- Ground level variations and thus pressure within the DMA
- Easily visible topographic features that can serve as boundaries for the DMA

Following points must be kept into mind while selecting DMAs:

- Maximum water demand
- Maximum population
- Maximum grievances

Each OZ, i.e., the area served by one overhead tank, should ideally comprise a single DMA, but in many circumstances, where the supply zone is exceptionally wide or the topography is extremely irregular, it is required to consider numerous DMAs in the same OZ. This is also necessary due to the scarcity of suitable sites for additional service reservoirs. DMAs are also designed within the OZ if high pressures develop in a section of the zone that needs to be isolated to form a separate DMA. When the pressure in such DMAs becomes too high, a PRV is sometimes employed at the inlet to reduce it.

Design of DMAs mainly consists of: (a) allocation of the nodes of the network called clustering; (b) identification of the pipes where isolation valves and flowmeters need to be installed, which is called vectorisation; and (c) performance evaluation of the partitioned network in terms of pressure and available demand.

A. Clustering of Nodes

The first step depends upon many factors such as its size (depending on geographical area, length of main pipes, number of consumer connections, etc.), topology considerations (number of feeds, the areas common with an adjacent district, boundary characteristics), type of consumption profile (variation of demand with space and time), and water quality considerations, etc. While partitioning, trunk mains shall preferably be identified and kept independent of DMAs. Such pipes should be distinguished from the network prior to DMAs identification. Traditionally DMAs are defined as per the sizing considerations. A DMA size typically consists of between 500 and 3,000 connections. The size of a DMA can also be smaller than the above-mentioned practice of water utilities depending on the purpose of work like identifying small leakages in the system. Several researchers have also used the hydraulic properties of water networks to identify clusters like nodal pressures to establish pressure zones and nodal demands to maintain demand uniformity in the DMAs. Identification of clusters can be done using engineering judgment-based rationale or it can also be achieved using sophisticated but freely available software tools like Nephi (www.gephi.org). After clusters identification, the interconnecting pipes (or boundary pipes) between the clusters should also be identified.

a. The clusters of the nodes can be achieved using the GIS tool. Following considerations are useful in clustering of nodes/pipes. Ideally the topography for a particular DMA should be more or less flat. Preferably for plain area the elevation difference within the DMA shall not be more than 5 m. This would ensure equitable pressure distribution within the DMA.
b. The DMA should be isolated with a minimum number of valves. This can be a problem with highly looped systems wherein a number of valves would have to be closed to isolate the DMA.
c. Natural boundaries such as rivers, streams, or other boundaries such as pumping stations, water treatment plants (WTPs), and pipelines laid on both sides of the roads, etc., should be used for demarcation.
d. Flow measurement should be used with a minimum number of flowmeters so as to maintain data accuracy.

B. Sectorisation Process

The second step of DMA design is to identify the open/closed status of boundary pipes. It is also called sectorisation of network. The basic idea of sectorisation is to close a few boundary pipes which are hydraulically less important. Generally, the cascading DMA should be discouraged, but if at all the terrain warrants the cascading DMA in such a situation the flowmeter can be added only at the start of cascading DMA. Conventionally, sectorisation is achieved by minimising the cost of flowmeters and isolation valves (cost aspect). This can be achieved using iterative methodologies like iteratively closing the different combination of pipes and finding out the best possible combination of open/closed pipes. This method may be useful for small networks where the number of boundary pipes is less. But in the case of large networks iteratively finding the best combination may prove to be cumbersome. In that case advanced heuristic optimisation tools like genetic algorithms (GA) may prove to be helpful to arrive at an optimal solution. Recently, researchers have been focusing on multi-dimensional DMA design problems which not only involves the cost aspect but also hydraulic aspect (minimum pressure requirements), customer satisfaction (available demand), quality aspect (water age) and reliability of network (resilience index). Such problems can be addressed using multi-objective heuristic optimisation tools.

If a separate inlet to DMA is not provided, then the distribution network becomes complex, and a lot of difficulties are faced in O&M of the system. This is the main reason for inequitable distribution of water. It is suggested to have multi-outlets (one for every DMA) from the ESR which is newly proposed for efficient design and smooth O&M of distribution system. If VFD pump is intended in the design, then the multiple inlets to different DMAs can be designed from the single outlet of the new or existing ESR. It should be decided on following considerations: (a) higher elevation within OZ so that staging height required is less, (b) central location so that DMAs can be planned in different directions and length of bulk line to DMAs reduces and (c) availability of land.

C. Performance Evaluation

The last step is the performance evaluation of the partitioned (after retrofitting) water network which helps to identify the changes occurring in the network after pipe closure activity. The performance of the partitioned network can be evaluated using statistical indicators like mean, minimum and maximum pressures inside individual DMAs. This step helps to identify the quality of partitioning. Also, this step helps in identifying the critical nodes where the pressure values may drop below the minimum requirement in case of abnormal conditions like pipe burst and create demand shortfall. The total demand requirement of individual DMAs can also be worked out in the performance evaluation stage so that in case of leakage or pipe burst, the search location can be quickly narrowed down to specific DMAs with abnormal demand and the repair works can be carried out quickly.

In case of formation of DMA in an existing network, isolation valves already available in the network should be considered. Zero pressure test can be carried out to check that in the isolated area computer model provides zero pressure.

12.12.2 Design of DMAs Using GIS

Size of DMA: DMA size is expressed in the number of properties. As per BIS IS 17482:2020, the size of a typical DMA in urban areas varies between 500 and 3,000 properties/metered connections. The size of an individual DMA may vary, depending on several local factors and system characteristics, such as:

a) the required economic level of leakage

b) geographic area and the /demographic factors (like, urban or rural, residential, commercial, industrial areas)

c) variation in ground level

c) previous leakage control technique (like, ex-waste meter districts)

d) individual water Agency/ Board preference (like, discrimination of service pipe bursts, ease of location survey)

e) Hydraulic conditions

DMAs in dense urban areas, like inner portion of cities, may be larger than 3,000 properties/metered connections, because of the high housing density. If a DMA is larger than 5,000 properties/metered connections, it becomes difficult to discriminate small bursts (like, service pipe bursts) from night flow data, and it takes longer to locate it. However, large DMAs can be divided into two or more smaller DMAs by temporarily closing the valves so that each sub-area is fed in turn through the DMA meter for leak detection activities. In this case, any extra valves required shall be considered at the DMA design stage.

Topography: DMA boundary is so fixed that it remains within normally available natural topographical features such as rivers, lakes, railway track, roads, etc.

Hydraulically Discrete: DMAs should be isolated from other adjoining DMAs for precision in measurements. For this, each DMA should have a single inlet for water and a district meter should be placed to monitor the inflow into it. Isolation can be achieved using the Isolation Valves. These valves should be initially set as closed and can be opened during any emergency/ pipe break cases, etc., Zero pressure test (Figure 12.15) should be carried out to validate hydraulically discreteness of the DMA.

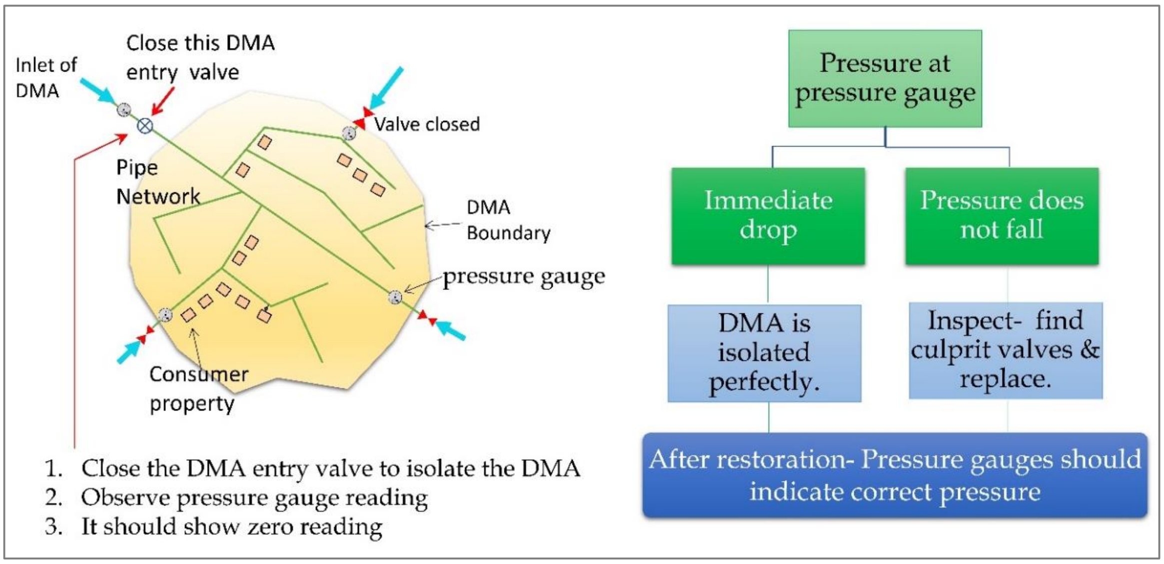


Figure 12.15: Zero pressure test

Cost of setting up DMAs: Cost of establishing DMAs will primarily depend upon the cost of isolation and metering equipment required. DMAs should be formed in such a manner that there will be minimum cost.

Slopes and elevation: DMA should be set up on uniform terrain. If a DMA has lot of uneven terrain conditions, supplying water would be difficult.

Establishing DMA Based on Number of Connections: GIS helps in measuring number of connections in OZs and DMAs. To start with the topology of the corresponding OZ is activated as shown in Figure 12.16. De-activated nodes and pipes are shown in red colour.

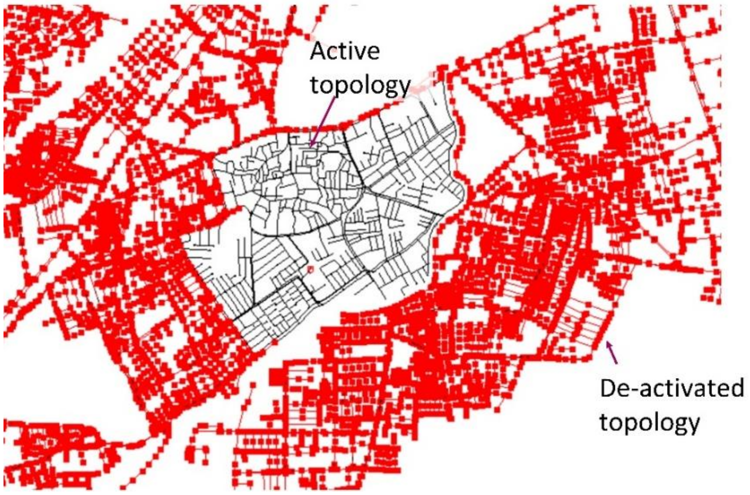


Figure 12.16: Activated network of OZ

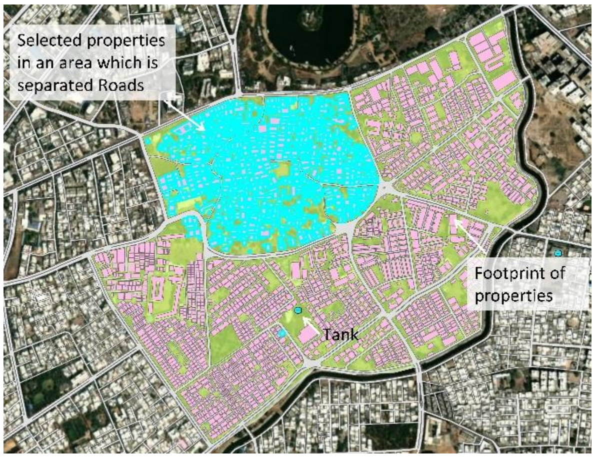
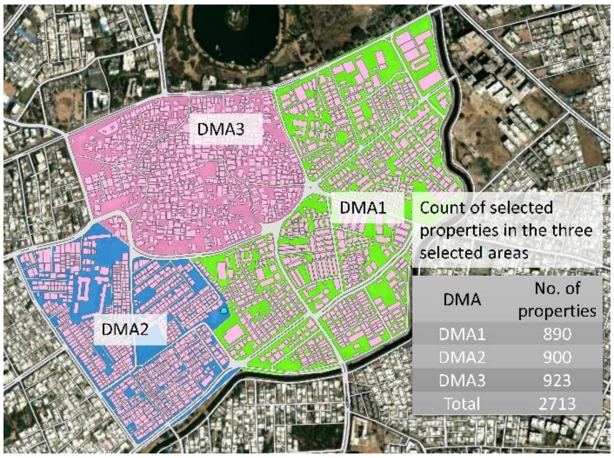


Figure 12.17: Selected nodes in one area bounded by the roads



DMA	No. of properties
DMA1	890
DMA2	900
DMA3	923
Total	2713

Figure 12.18: Nodes in all the three areas

The extent of OZ (Figure 12.16) seems to be large and hence about three DMAs are expected. Depending upon the size of the OZ, the number of DMAs to be created shall be anticipated. To begin with, three DMA areas in the activated OZ are selected by colour coding. It is required to compute the number of properties in these three selected areas. Shape file of the three areas is created. Using GIS tool, the number of properties in one of the areas (Figure 12.17) are selected and counted. In this way, the number of properties in all the three areas (Figure 12.18) are counted.

Alternatively, one field is added in the *attribute table* of the shape file of properties in all the three areas. The shape file of the properties is then *spatially* joined with the shape file of the three areas. GIS software adds the labels of the corresponding areas (say, DMA1, DMA2 and DMA3) to the added field of the respective polygons of the properties. Thus, all the polygons (footprints) of the properties get labelled. By summarising on the added field, the count of the number of properties in all the three areas (Say within DMAs) is determined which is shown in Figure 12.18.

Knowing the connections per property from the billing data, the number of connections in all the three areas (DMAs) is computed. If the number of connections is less than 3000, the boundary of the selected areas can be suitably increased.

One Inlet for Each DMA

If a separate inlet to DMA is not provided, then the distribution network becomes complex, and a lot of difficulties are faced in O&M of the system. This is one of the reasons for inequitable distribution of water. If the proposed service tank is new and if the network is designed to feed by gravity, it is recommended to have multi-outlets (one for every DMA) from the ESR for efficient design and smooth O&M of distribution system.

However, if the service tank exists then all DMAs should be fed by branch pipelines starting from outlet of ESR in OZ (Figure 12.19).

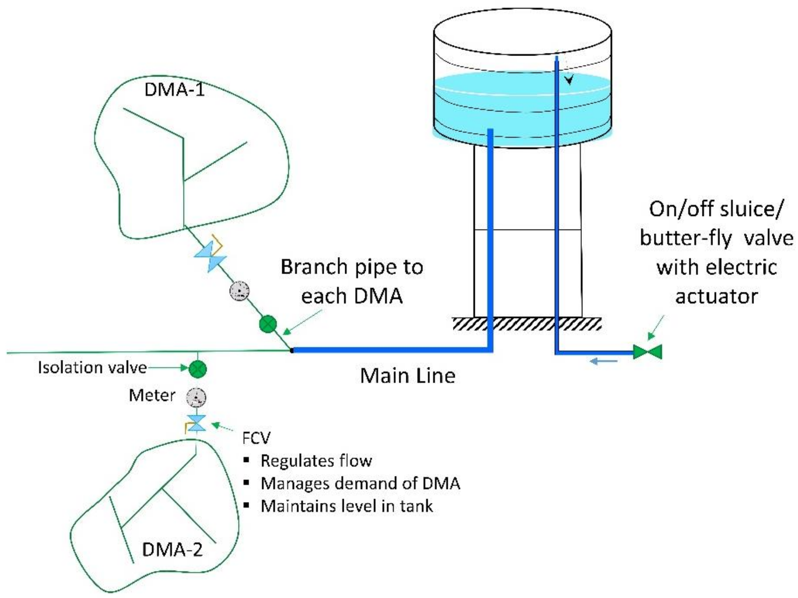


Figure 12.19: Separate branch pipeline to each DMA

From these pipelines consumer connections should not be given. Each DMA should have only one inlet. By this arrangement and by limiting the size and boundary of DMAs equitable distribution of water as per designed nodal demands with designed residual head can be achieved.

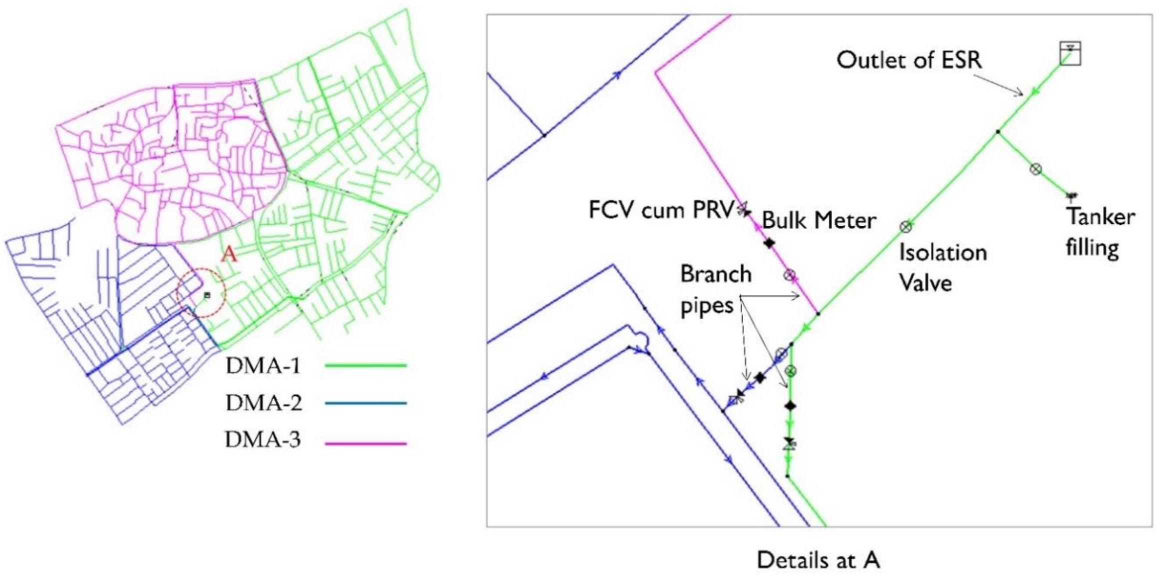


Figure 12.20: Separate branch pipes for entry of each DMA

The arrangement of separate branch pipes for entry to each DMA is shown in Figure 12.20.

12.13 Pipelines on Both Sides of Roads

Currently, many cities are being transformed into Smart Cities. Smart cities require smart roads which can monitor the traffic and signals. However, not only the smart roads but the concrete roads having width of 6m or more also need water pipelines on both the sides. Boundaries of the OZs and their DMAs are so located topographically that its spread remains within normally available topographical features such as rivers, lakes, railway track, bigger width roads, etc. The exercise of planning the pipeline on both sides of the smart roads can be done using GIS and the network software.

It is necessary to lay pipelines on either side of the concrete road so that while giving house connection, the road is not required to be damaged. The method for concrete roads having width less than 6m is to insert the ducts intermittently in the body of the roads so that service connection pipes can be laid through it. For concrete roads having width 6 m or more, pipes are to be laid on either side of the road. This can also be done economically while deciding the boundary of DMA.

12.14 Pressure Management

Pressure management is important in water distribution for two reasons- (a) Equitable Flow and Pressure, (b) Improving nodal pressure to 21 or 15m as the case may be and (c) Reducing water loss due to leakages due to controlling pressure.

12.14.1 Equitable Flow and Pressure

OZs and DMAs should be planned and designed by the *whole-to-part* approach. They should be planned with 100% consumer metering along with telescopic tariff so that demand management is possible. The whole-to-part approach is discussed below by use of PRVs and FCVs, however, PRVs should be used only when required. PRVs are mainly required in hilly cities or where the OZ or DMA is at lower elevations. The maximum head that comes on pipes in the distribution system is the difference between FSL of ESR and minimum ground elevation in the OZ of that ESR. Class of pipe should be so chosen that working pressure of that pipe is more than the maximum head coming on the pipes in that OZ.

Equitable distribution of water with equal pressure is a need for a 24×7 system. It is achieved by whole-to-part approach, in which three stages are involved: (i) from MBR to ESRs; (ii) from ESR to DMAs; and (iii) Equitable Pressures within DMA.

(i) From MBR to ESRs: In this stage, MBR supplies water to different ESRs as shown in Figure 12.21.

Equalisation of residual head in ESRs: In an ideal design of water network, residual heads at all the service tanks should be the same as the minimum required ones. Detail method of design of transmission main and achieving equal residual head at FSLs of ESRs has been discussed in the Chapter 6. By this method, every ESR draws water as per the designed demand of that ESR.

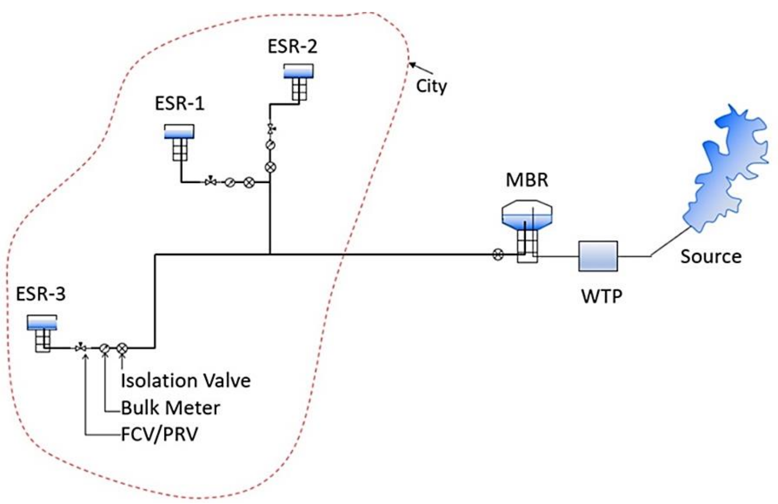


Figure 12.21: Arrangement of valves at inlet of every ESR.

In addition to equalisation of heads at ESRs, the inlet of each tank should be provided with isolation valve, bulk meter and then FCV/PRV. In normal case, FCV is sufficient because, while controlling the flow, the pressure is proportionately reduced. However, in situations having steep slopes such as in hilly city, both the PRV and FCV are required. In such situations, the sequence in the direction of flow would be isolation valve, bulk meter, PRV and then FCV (Figure 12.21). Precautions should be taken to locate the bulk meter prior to PRV and FCV. The FCV is so set that the inflow to tank would be as per the demand of the OZ served by that tank.

(ii) From ESR to DMAs: In this case, ESR supplies water to different DMAs as shown in Figure 12.19. A bulk meter and FCVs are required for DMAs, the sequence of the valves should be as discussed above. FCV with solenoid may be installed at the entry point of the DMA. FCV with solenoid is provided at the entry point of each DMA. There is an inherent mechanism like router (transmitter) which is connected with the programmable logic controller (PLC) of the supervisory control and data acquisition (SCADA). This arrangement automatically adjusts the flow during peak and non-peak hours.

(iii) Equitable Pressures within DMA: The topography of DMA should be preferably flat which will ensure equitable pressure distribution within the DMA. For this purpose, the lowest and highest elevation points should be found out. The elevation difference between these two points should normally be 4 to 5 m.

12.14.2 Improving nodal pressure to 17-21 m

Hydraulic Model of 24×7 Water Supply Using VFD Pump

The residual nodal pressures are recommended to be in the range of 17-21 m at critical node of the distribution network. However, in India, many service reservoirs have staging height less than 12 m and may not be able to meet the pressure requirement by any change in pipe sizes. Hence, for 24×7 water supply instead of constant speed pumps, direct feeding of networks is recommended using VFD operated pumps adopting smart control philosophy. A new network can be easily designed with VFD pump installed at clear water sump of WTP to pump water directly into the distribution network. However, for an existing network with less staging height of service reservoirs, installation of VFD pump on outlet of such service tank is desirable to increase the residual nodal pressures to 17-21 m. If any of the DMA is found to be heavily leaking due to high pressure, then RPM of VFD can be controlled by a suitable frequency or the PRV can be operated to regulate the pressure between 17 m to 21 m. The details of direct feeding by VFD pumps are mentioned in Chapter 5 "Pumping Stations and Pumping Machinery" in Part A of this manual.

In India, WDNs are usually designed to achieve a residual head of 7 to 12 m for supply to single- and double-storeyed buildings, respectively. Considering the supply of drinkable water from consumer taps without the requirement of storage and household treatment as far as possible, it is recommended to have a residual pressure of 17 to 21 m at all nodes throughout the day. A combined pumping and gravity network are preferable by constructing a service reservoir which provides sufficient storage to take care of fluctuating demand and allow the pumps to operate at constant head. In this case, pumps are selected to have their duty point in the best operating zone (most efficient zone) and service reservoir incidentally acts as a break pressure tank, allowing the network on downstream of the reservoir to work as gravity-fed network. However, with the increase in residual head requirement, a combined pumping and gravity network may not be feasible in some cases due to restriction on the height of reservoir. Further, maintaining higher residual pressure in an existing network with the existing service reservoir may not be possible. In such cases, a direct feeding to distribution network through pumps is desirable. As the demand and pressure requirements in the network are varying throughout the day a constant speed pumps should not be used as it may not operate in best efficient zone for most of the time in a day. VFD pumps are suitable for feeding distribution networks directly, and hence recommended. VFD pumps can work at different speeds allowing the pumps to operate more efficiently in different period of varying demand and pressures. A comparison of complete gravity (or combined pumping and gravity feed) and direct feed networks is shown in Table 12.2.

Table 12.2: Comparison of gravity feed and direct feed networks

S. No.	Gravity Feed Network	Direct Feed Network
1	The distribution network is connected to ESR or hill service reservoir.	The distribution network is connected directly to pump water into the distribution pipelines. Each pump group may feed 2-4 DMA.
2	Variation in network demand is covered by buffer storage volume in the service reservoir.	Variation in network demand is controlled through pump speed/output.
3	Level based, fixed speed of pumps	Demand based, pressure control at variable speed of pumps by defining system head to meet required pressure at critical points.
4	High variation in residual pressure at critical point (highest elevation) in the network.	Residual pressure in network at critical point is maintained in narrow band.

S. No.	Gravity Feed Network	Direct Feed Network
5	Comparatively high energy consumption.	Most energy efficient operation.

As direct feeding distribution networks using VFD pumps have many advantages like, energy saving and assured 24×7 continuous supply with required residual pressures at critical points, it is recommended to adopt this method in metro and major cities where the distribution network of proper pipe materials is available, and electricity is continuously available through express feeders.

Principle of VFD Pump

A VFD is a motor controller that drives an electric motor by varying the frequency and voltage supplied to the electric motor. The frequency can be changed using electronic drive circuit which then alters the rotational speed of pump. Frequency (or Hertz) is directly related to the motor's speed denoted by the revolutions per minute (RPMs). In other words, the higher the frequency, the faster the RPM of the pump.

When VFD pump starts, AC wave is generated. Frequency of the AC wave is defined as the number of cycles of signal that took place in a second. Frequency is measured in Hertz (Hz). For example, if 50 complete cycles are produced in one second, then the frequency of the given wave is 50 Hz. In India, the value of frequency is 50 Hz. This value is standardised and widely used around the world as the standard frequency for the alternating current (AC). Relation between the rotation, frequency, and the number of poles is given by,

$$N = RPM = \frac{120f}{P} \qquad \text{(Eq. 12.10)}$$

where N = Rotation/minute (RPM); P = Number of poles; and *f* = Supply frequency.

Pump Head-Discharge Curve

The pump curve denotes the relationship between rate of fluid flow and head for the pump itself. Flow is on X-axis and the pressure head generated by pump is shown on the Y-axis as shown in Figure12.22. Starting point of the pump curve is at the point of zero flow, at which the pressure head generated by the pump is called shutoff head. It descends and the lowest point is at maximum flow rate. Even though a pump can operate over a long range of discharge and head, its efficiency will not be same at different points. Best operating zone is defined as the zone over which pump efficiency is more than some desirable efficiency. A pump is selected to operate in best operating zone.

Effect of VFD on Pump Performance

The curves indicating stable pump operation range as recommended by the manufacturer shall be used. It is an envelope formed by (Qmin - Qmax) conditions at minimum and maximum speed. Qmin is the minimum flow (LPS or LPM or m^3/hr) at minimum rated speed Nmin (in RPM), and Qmax = maximum flow (LPS or LPM or m^3/hr) at maximum rated speed Nmax (in RPM).

Affinity laws: Affinity laws are used to calculate head or power consumption in centrifugal pumps when changing speed or wheel diameters. Thus, for the same impeller diameter, when the pump speed changes, flow rate is directly proportional to the speed, so also the head is directly proportional to the square of the speed. If a pump delivers a discharge Q1 at a head H1 when running at speed N1, the corresponding values when the same pump is running at speed N2 are given by the similarity (affinity) laws:

$$\frac{Q_2}{Q_1} = \frac{N_2}{N_1} \qquad \text{(Eq. 12.11)}$$

$$\frac{H_2}{H_1} = \left(\frac{N_2}{N_1}\right)^2 \qquad \text{(Eq. 12.12)}$$

$$\frac{P_{i2}}{P_{i1}} = \left(\frac{N_2}{N_1}\right)^3 \qquad \text{(Eq. 12.13)}$$

where, Q = discharge (m^3/s, or l/s), H = pump head (m), N = pump rotational speed (rpm) and P_i = power input (HP, or kW).

The System Head Curve

The system head curve represents relationship between head and flow of the distribution pipe network. It shows how much head is required to push flow rate through the pump and into the distribution system. As the head loss in pipe network increases with increase in flow, the system head curve shows increase in head due to increase in water flow through the pipework. The total head, H that the pump delivers includes the elevation head and the head losses incurred in the system. The friction loss and other minor losses in the pipeline depend on the velocity of the water in the pipe, and hence the total head loss can be related to the discharge rate.

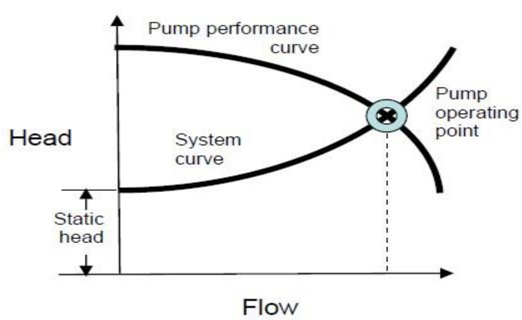


Figure 12.22: Operating point of pump

System head for a simple pumping main can be defined in mathematical form as

$$H_s + (R * Q^n) \qquad \text{(Eq. 12.14)}$$

where, H_s denotes static lift, i.e., difference of elevation head at critical point in the pipeline and lowest suction level; and R is resistance of pipe in the head loss equation. Value of n can be 1.852 or 2.

Selection of Pump

A proper pump is selected by considering both the system curve and the pump curve. When combined, pump curve intersects the system head curve (Figure 12.22) which is called the operating point.

System Head Curve for Complex Network: System head curves can be easily developed for a system with tanks on both the suction and discharge side of the pump as discussed and shown above in Figure 12.22, which is termed as an open system (Walski et al. 2010). But most of the distribution system do not have tank (floating tank) on the downstream side of the pump, which is called a closed system. Usual methods of creating system head do not work on such (closed) systems. In such situation hydraulic model is used to create system head curve.

For a given pump group, there can be several system curves for a particular network. However, a unique system head-capacity (H-Q) curve can be plotted by analysing residual pressure requirements at critical point in the network.

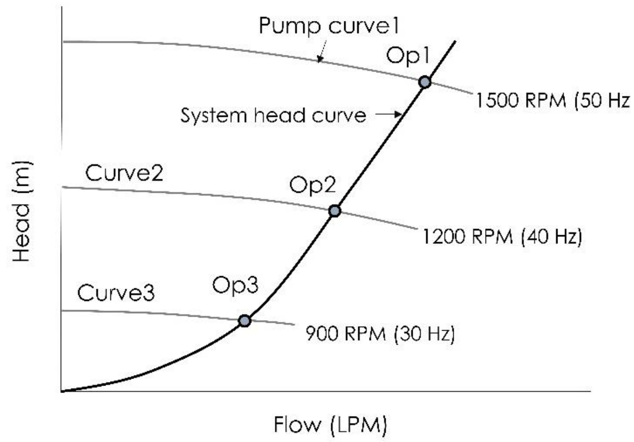


Figure 12.23: Attainting operating points on system curve

Figure 12.23 shows a system head curve and three pump curves for different rotational speeds of single pump system with four number of poles. There is one VFD for this pump. In Figure 12.23, pump curve 1 is for frequency of 50 Hz with operating point Op1 for maximum demand, say in peak hour. When the demand reduces, the frequency is lowered to say 40 Hz and the operation point shifts to Op2. At midnight, the demand is least, and the pump speed further reduces to, say 30 Hz and the operating point shifts to Op3. In this way, the change in demand is dealt with alteration in the rotational speed of the pump.

A case study of hydraulic modelling of one OZ of Ayodhya City using VFD pumps is discussed as shown in **Annexure 12.4**.

Excessive Nodal Pressures

The design of OZ is carried out for a nodal pressure of 21 m. If the nodes in the network have more pressures, then there is need to curtail the pressures in nodes. For example, as shown in Figure 12.24, due to low elevations in the terrain, some nodes in DMA 2 have nodal pressures even more than 40 m. Authority encounter the 'tail end problem' which causes a major obstacle in ensuring equal water supply at every tap.

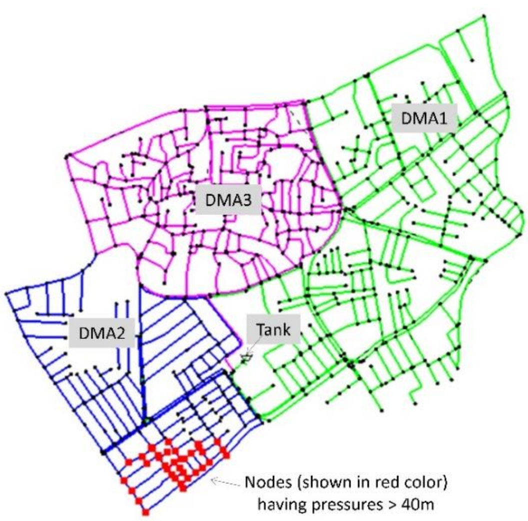


Figure 12.24: Higher nodal pressures

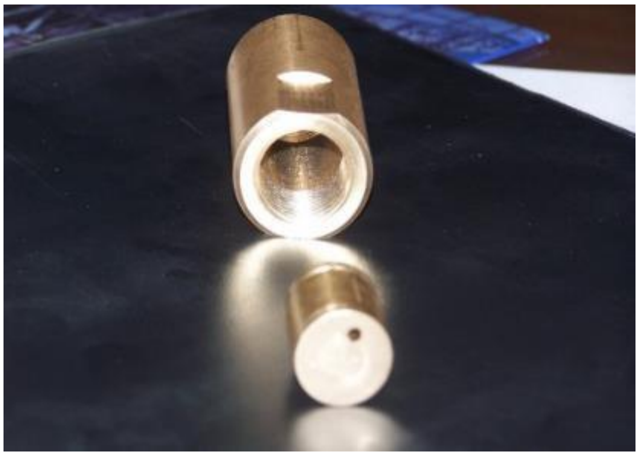


Figure 12.25: Pressure and Flow Rate Reducing Valve [PFRV]

To resolve this problem, it is proposed to install (Figure 12.25) pressure and flow rate reducing valve (PFRV) in the WDS. PFRV consists of a series of orifice plates, placed at designed spacing, to meet the requirements of a specific site. The water between these plates gets churned and helps in achieving the desired goal.

PFRV has the potential to simultaneously reduce both water pressure (dynamic + static) and flow rate of a flowing water in almost equal proportions, and helps in resolving the 'tail end problem', saving consumers from water splashes while reducing water wastage.

This PFRV shall be installed in a water distribution network, at the following places:

- To achieve a solution to the 'tail end problem' in vertical and horizontal planes, this PFRV shall be installed, just after every connection/ferrule point.
- In hilly areas, to achieve water availability in horizontal water mains, this PFRV shall be installed just below a junction point, where the vertical and horizontal water main meets.

For efficient working of a PFRV, the upstream water pressure should be in the range of 2 - 3 kg/cm^2 to receive a water flow rate of about 05-08 L/minute. However, the PFRV can perform efficiently at any site where the upstream pressure is about 03-14 kg/cm^2 or even more.

This PFRV does not have any spring or movable part and hence, considered to have a larger life expectancy than a PRV. The outflow from a PFRV cannot be manipulated, and it can easily be repaired as and when required.

Pressure Management in Hilly Areas

Application PRVs and direct acting PRVs: Usually, hilly areas contain the following landscape features: (1) they are far away from the water source and urban areas, (2) they contain more dispersed WDNs, and (3) the terrain elevations in the house group vary greatly. In hilly areas, it is more difficult to divide the water supply system reasonably than it is in flat areas. A pressure management is necessary in the water supply systems of the hilly areas.

Distribution system is designed to provide water to consumers at some agreed level of service which is often defined as a minimum level of pressure at the critical point which is the point of lowest pressure in the system. This minimum pressure is 21 m water head.

In hilly cities, water pressures are huge, about 300 m at tail ends. Hence, there is a need to manage the pressures by reducing them. Pressures are reduced by the techniques of "Fixed outlet pressure control". It involves the use of PRV. This is possibly the simplest and most straightforward form of pressure management as it involves the use of a PRV with no additional equipment.

Unfortunately, in most of the parts of the distribution system of the hilly city, layouts of the house properties are vertical. Hence, many PRVs are required to reduce the residual nodal pressures from 300 m to about 60m.

The pressure surface in all the areas without PRV is shown in Figure 12.26 and with PRV in Figure 12.27.

From Figures 12.26 and 12.27, it is observed that if PRVs are not installed, then there is huge nodal pressure of 374 m, however, when PRVs are considered, the nodal pressure is still in the range of 20 to 98 m, however, for managing this pressure everywhere we require a large no. of PRVs which is practically not possible and uneconomical, hence Direct acting PRVs play very important role to further reduce the residual nodal pressure to 21m.

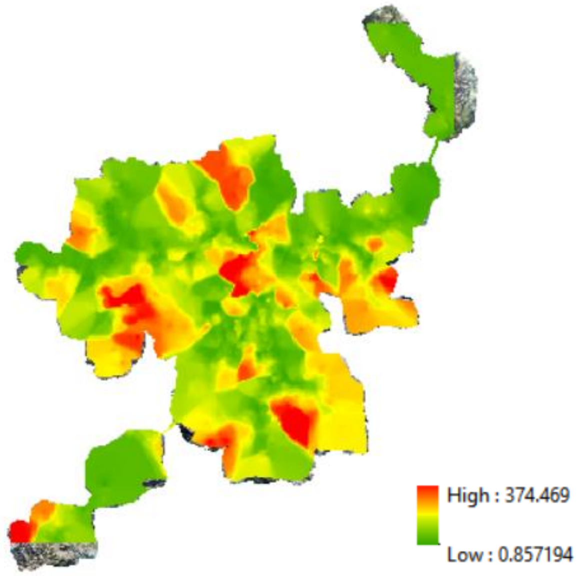


Figure 12.26: Without PRV: Pressures in hilly area

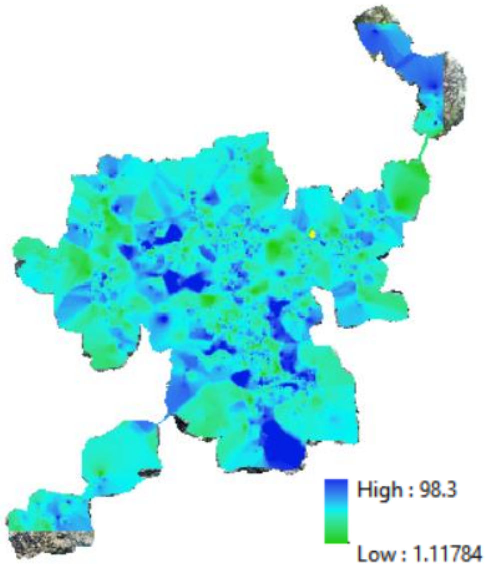


Figure 12.27: With PRV: Pressures in hilly area

Direct Acting PRVs: Direct acting PRVs are used to reduce residual nodal pressures to 21 m. It is recommended that every connection should have one direct acting PRV. These valves are used in high-rise buildings to control pressure fluctuations between floors. These valves are also used in municipal water systems at service connections in a high-pressure distribution zone.

12.14.3 Reducing Water Loss by Controlling Pressure

Water loss is represented by the NRW which is defined as,

$$NRW = \frac{\text{Water put into system - Total water billed}}{\text{Water put into system}} \text{ x } 100 \qquad (12.15)$$

The reduction of NRW is a crucial step to improve the financial health of water utilities and to save scarce water resources. The percentage of physical losses is influenced not only by the deterioration of piped network, but also by the total amount of water used, system pressure, and the degree of supply continuity.

To a large extent, the level of NRW is an indicator of how well a utility is managed. Many cities have NRW values, more than the national average of 31%. They should target low NRW and, accordingly, chalk out the programme for it.

Impacts of NRW

In many water utilities, there are high levels of NRW which leads to low levels of efficiency in terms of financial economy and redressal of complaints. When a utility's product (treated water) is lost, water collection, treatment, and distribution costs per unit of volume increases, water sales in terms of volume and amount decreases, and to resolve this situation, substantial capital expenditure programmes are often promoted to meet the ever-increasing demand. In short, the utility enters a vicious cycle (Figure 12.28) that does not address the core problem.

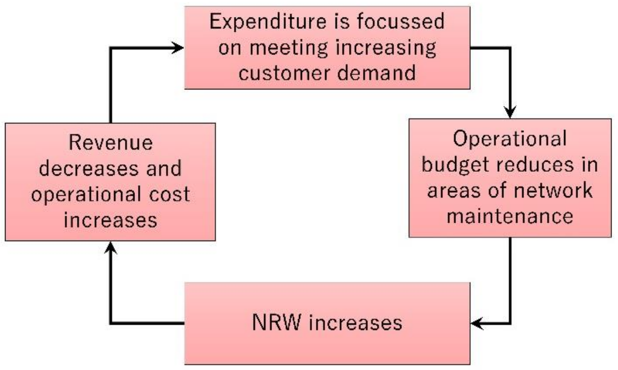


Figure 12.28: Vicious Circle of NRW

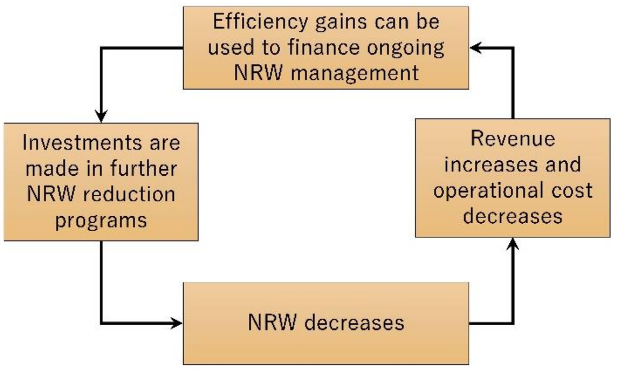


Figure 12.29: Virtuous Circle of NRW

The challenge for these utilities is to turn this vicious cycle (Figure 12.28) into a virtuous cycle (Figure 12.29), which will lead to low levels of NRW and, therefore, substantially improved efficiency. In most cases, many municipal organisations and ULB's focus on vicious cycle for reduction of NRW instead of focusing on core problem. This is happening with many water utilities.

12.14.4 Water Audit

For effective control of water losses, NRW of every DMA is to be determined by dividing OZs. A city is divided into a number of OZs which are further divided into number of sub-zones called DMAs. Each DMA is then critically studied for different demand patterns, leakages and unaccounted for water. Thus, the problem is divided into sub-problems and effective control measures are taken to provide an effective solution for each sub-problem to solve the problem in total.

Water audit identifies how much water is lost and the loss of revenue against the same. The objective of water audit is to help the utility select and implement programmes to reduce distribution system losses. Water audits should be performed annually to help managers to adjust priorities, monitor progress, identify new areas of system losses, and establish new maintenance goals. A water audit followed by leak detection programme can help water utilities reduce water and revenue losses and make better use of water resources. Details of Water Audit are discussed in Chapter 11 of Part B of this manual.

Water Balance

Effective water management scheme aims at understanding the standard water balance and minimising/avoiding NRW. A standard water balance is shown in Table 12.3. Total input of water in a water distribution network can be divided into two parts, (a) Revenue water and (b) NRW.

Components of Water Balance: They are as below.

A. **Authorised consumption:** It includes the volume of metered and/or unmetered water taken by registered customers. Authorised consumption includes water required for firefighting and training, flushing of mains, street cleaning, watering of municipal gardens, public fountains, building water, etc. These may be:
   - (i) Billed authorised consumption: It includes consumption of the consumers who are metered and billed and are producing revenue. It also includes billed unmetered consumption.
   - (ii) Unbilled authorised consumption: Though consumption, in this category, is legitimate but it is not billed and therefore do not produce revenue. These also include unbilled unmetered consumption.

Table 12.3: Standard Water Balance*

<table>
<tr><td rowspan="9">System Input Volume</td><td rowspan="4">Authorised Consumption m^3/year</td><td rowspan="2">Billed Authorised Consumption m^3/year</td><td>Billed Metered Consumption (Including water exported)</td><td rowspan="2">Revenue Water m^3/year</td></tr>
<tr><td>Billed Unmetered consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorised Consumption m^3/year</td><td>Unbilled Metered Consumption</td><td rowspan="7">NRW m^3/year</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="5">Water Losses (NRW) m3/year</td><td rowspan="2">Commercial Losses m^3/year</td><td>Unauthorised Consumption</td></tr>
<tr><td>Metering Inaccuracies and Data handling error</td></tr>
<tr><td rowspan="3">Physical Losses m^3/year</td><td>Leakages on Transmission and Distribution Mains</td></tr>
<tr><td>Leakages and Overflows at Utility's Storage Tanks</td></tr>
<tr><td>Leakage on Service Connections up to point of Customer metering</td></tr>
</table>

** Standard Wable Balance table modified for indian conditions can be referred from Table 11.1 in Part B of this manual.*

B. Water losses: Water losses comprise of the commercial losses and physical losses.

(i) Commercial losses: These include unauthorised consumption such water theft through illegal connections. It also includes inaccuracies associated with customer metering as well as data handling errors made by the meter readers and computation errors at the time of billing.

(ii) Physical losses: It comprises of physical leaks on transmission mains and distribution mains, losses due to overflow in tanks and leaks on service connections especially at the ferrule point.

12.15 Estimating Losses

Once DMA is established by fixing flowmeter at entry point of each DMA, it becomes a tool for monitoring the NRW. NRW has two components, physical and commercial losses. Both these components can be monitored. NRW in DMA is computed by the following equation,

$$\text{DMA NRW} = \text{Total DMA Inflow} - \text{Total DMA Consumption} \tag{12.16}$$

12.15.1 Estimating Physical Losses

With bulk meter installed at the entry point of DMA, total DMA inflow is measured. If 100% metering is made within DMA, total DMA consumption would be summation of consumer meter measurements for the period in which calculations are made.

Physical losses within DMA are due to the leaks on main pipes and leakages through the consumer house connections. Leaks from the main pipes would be continuous for the whole 24 hours of the day; whereas leaks from consumer connections fluctuate due to consumer's demand at peak hours and are minimum at night. Therefore, leakages during night should be monitored. A flow pattern in a typical DMA is shown in Figure 12.30.

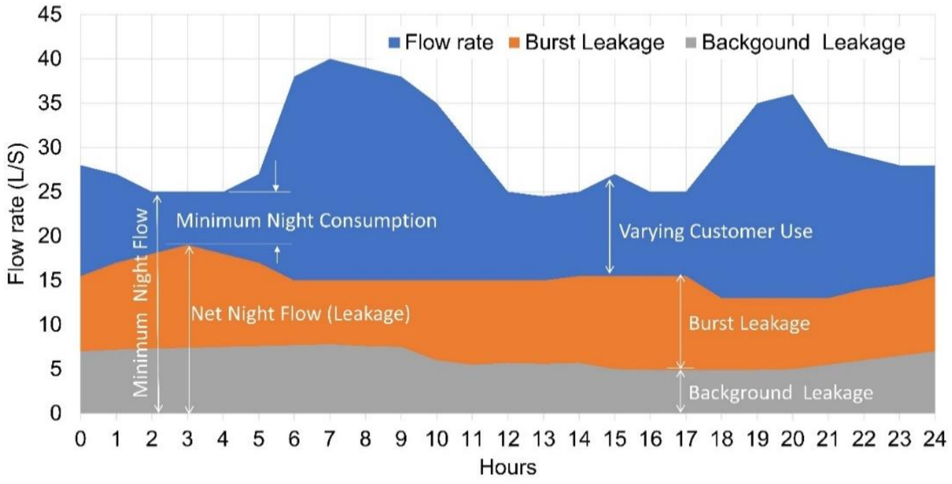


Figure 12.30: Flow pattern in a typical DMA

Inflow to DMA is measured continuously by the bulk meter which is installed at the entry point of DMA resulting the curve shown in blue in Figure 12.30. Its minimum night flow (MNF) is noted which is at night. If the legitimate night flow (LNF) is subtracted from the MNF, net night flow (NNF) can be computed which is as below,

$$NNF = MNF - LNF \qquad (12.17)$$

12.15.2 Estimating Commercial Losses

Since total NRW = Physical Loss + Commercial loss, Commercial loss is computed by,

$$\text{Commercial losses} = NRW - NNF \qquad (12.18)$$

12.15.3 Leak Repair Programme

Bursts can be identified by the variation in MNF over longer period, say 180 days. A typical such variation in a DMA is shown in Figure 12.31. These variations in night consumption can be observed and then can be identified and repaired.

Reported bursts are visible leaks and are also removed in reasonable time by ULB. However, small leakages do not come to surface and cause increase in NRW and contamination. These invisible leakages appear and are shown as background leakages in Figure 12.30.

Unreported bursts can also be detected, as shown in Figure 12.31. One such unreported burst appears on day 45, and since it is not removed, the losses are continued. Another unreported burst occurs on day 67. When both the unreported bursts are removed, NRW level is brought down as shown in graph.

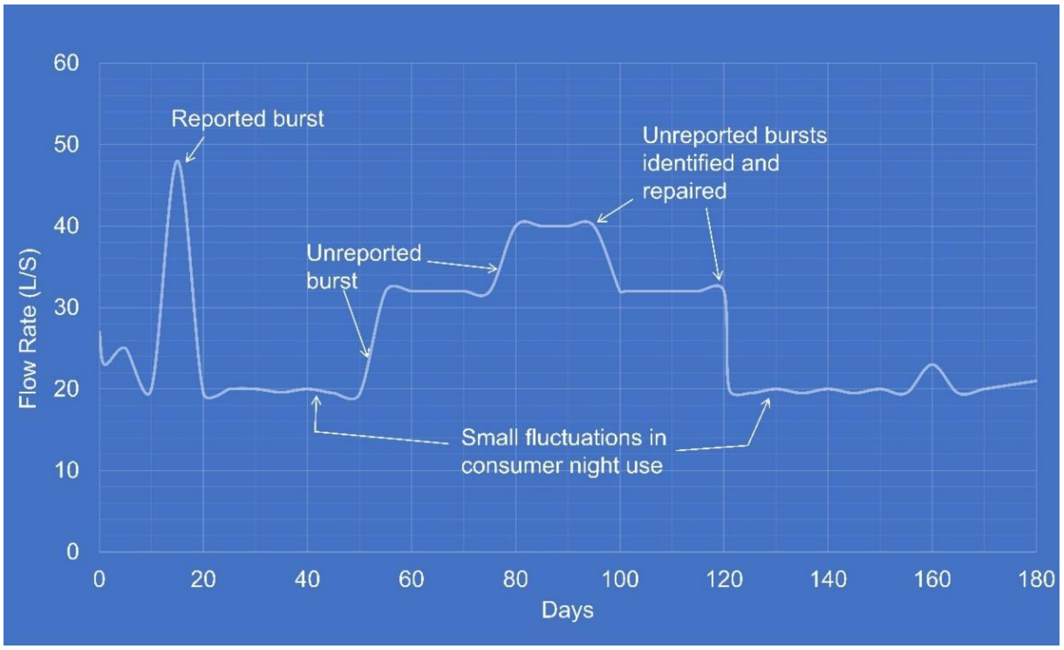


Figure 12.31: Fluctuation in MNF over 180 days

12.15.4 SCADA Attached to DMA

DMA meter can be connected to the SCADA system. SCADA is a computer system used for collecting and analysing real time data. SCADA systems, when connected, are used to monitor variation in MNF and, hence, can be used to identify the leaks and bursts in the system.

12.16 DMA management

As soon as DMA is established, initial values of NRW and NNF should be recorded. As shown in Figure 12.32, NRW values generally increases with time. Operator should fix the intervention limit. When NRW reaches this limit, the task of NRW reduction is taken up. NRW is lowered to its base level. As time moves on, the value of NRW increases again. Operator has to again bring NRW to its base level. If frequency of intervention increases rapidly, then the pipe replacement should be made.

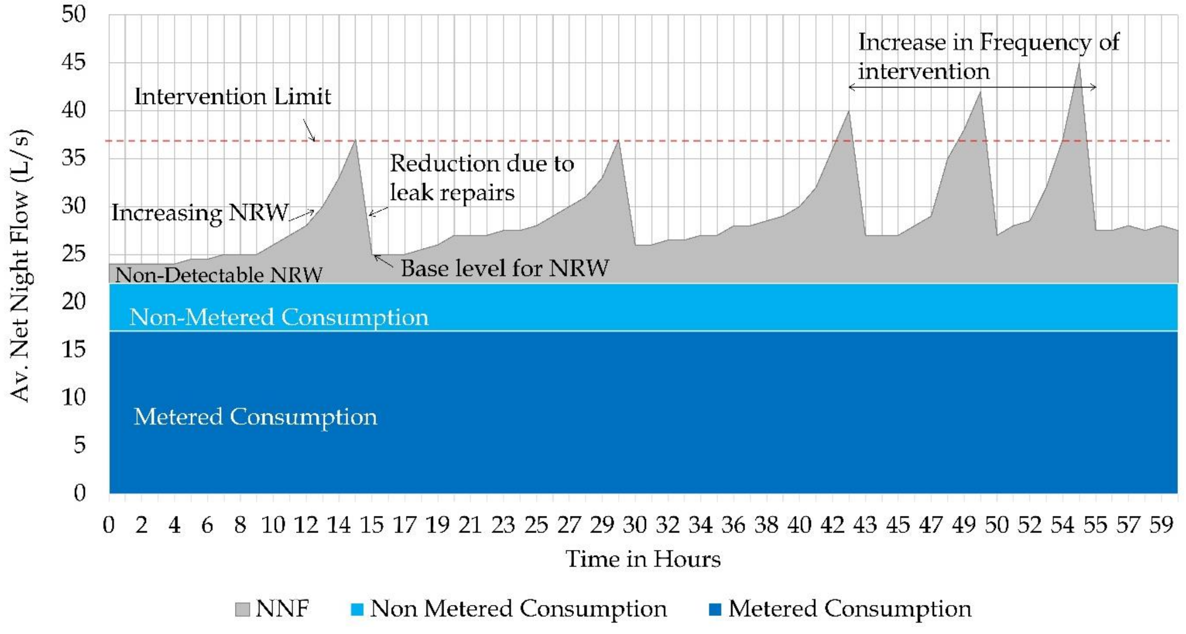


Figure 12.32: Increase of NRW and intervention with respect to time.

12.17 Step Test

Step test is generally used to compute NRW within DMA. The name 'step' in this test has come from the appearance of graph which resembles with 'steps of staircase' (Figure 12.34). When the rise of step is too large, then corresponding section has more NRW. If size of DMA is large, it should be split into smaller sections called sub-DMA. Step tests narrows search for leakage spot. This test is laborious and time consuming.

DMA is divided into small sections by closing valves. Each small section is shut off at night (www.matchpointinc.us). Before any valves are closed, the MNF is recorded. It is called "START" MNF value. Then, as each valve is closed systematically, this is called a "STEP" and the new MNF is recorded. The difference between the start and the new flow is your "STEP" value which approximates NRW value.

Illustrative Example: Network of DMA is shown in Figure 12.33. Using isolation valves, nine segments are created, which are shown in Figure 12.33.

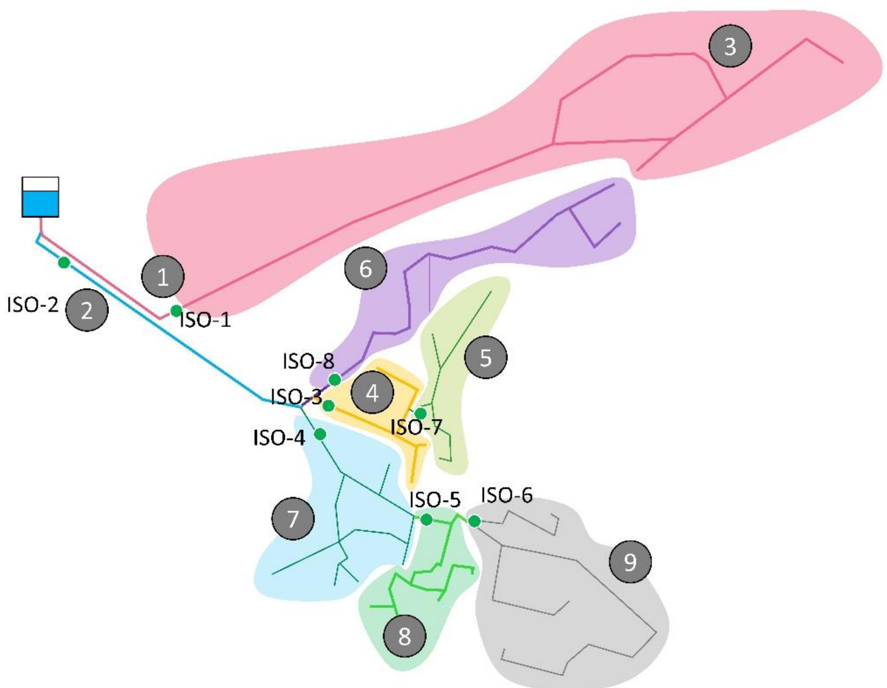


Figure 12.33: Nine segments (numbers shown in black circles) in DMA created by isolation valves (Mulay and Bhole 2021).

Test Procedure: The test started at 12 PM (00:00 hours) for determination of MNF. As the name implies, it is to be conducted at night when legitimate consumption is less.

1. Before closing valves, Start MNF should be noted which is 48.43 L/M.
2. Isolation valve, ISO-6 is closed and thus, Segment 9 is closed, hence, flow in it is zero. The bulk meters reading (42.72 L/M) at the outlet of the ESR is taken. The results are shown in Column 4 of Table 12.4.
3. Isolation valve, ISO-5 is now closed in addition to ISO-6 and thus, Segments 8 and 9 are now closed, hence, flow in these two segments is zero. The bulk meters reading (which is 37.79 L/M) at the outlet of the ESR is taken.
4. The above steps are repeated till last Segment 1 is closed.

5. The valves are then systematically opened in the reverse way and the MNF is again recorded. Initially, flow in Segment 1 is recorded.
6. The process of opening valves is repeated till all the segments are opened one by one and each time the smart meter (bulk meter) readings (MNF in litres per minute) are noted.

Table 12.4: Sequence of valve operations and computation of NRW

SN	Section Closed	Valves to be closed	MNF observed at bulk meter at outlet of ESR (L/M)	MNF (L/m) Calculated by deducting present reading from previous	Segment	Connections	Legitimate flow, LNF = Col.7 * 0.08333 (L/M)	NRW= MNF-LNF (Col 5 - Col 8) (L/M)	NRW (%) = (Col 5 - Col 8)*100/ Col 5
1	2	3	4	5	6	7	8	9	10
0	Nil	All opened	48.43						
1	9	6	42.72	5.71	9	39	3.25	2.46	43.1
2	9+8	6,5	37.79	4.93	8	34	2.83	2.10	42.5
3	9+8+7	6,5,4	28.20	9.59	**7**	**36**	**3.00**	**6.59**	68.7
4	9+8+7+5	6,5,4,7	25.20	3.00	5	23	1.92	1.08	36.1
5	9+8+7+5+6	6,5,4,7,8	18.90	6.30	6	34	2.83	3.47	55.0
6	9+8+7+5+6+4	6,5,4,7,8,3	13.80	5.10	4	37	3.08	2.02	39.5
7	9+8+7+5+6+4+3	6,5,4,7,8,3,1	9.86	3.94	3	34	2.83	1.11	28.1
8	9+8+7+5+6+4+3+2	6,5,4,7,8,3,1,2	5.86	4.01	2	29	2.42	1.59	39.6
9	All	All closed	0	5.86	1	38	3.17	2.69	46.0
			Total	48.43		304			

The results of MNF are plotted against the time as shown in Figure 12.34. Also, MNF (NRW) in each segment is computed.

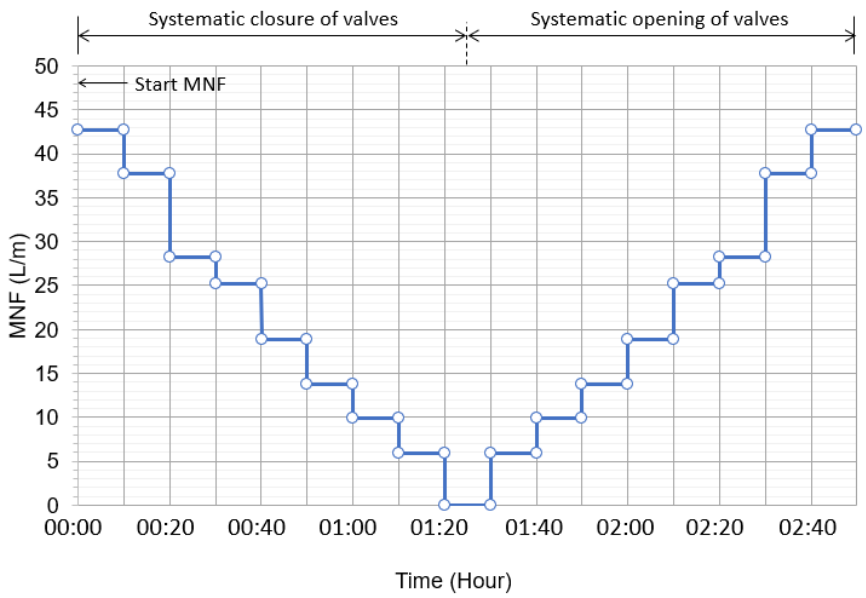


Figure 12.34: Values of MNF Vs. Time

Before conducting the STEP test, LNF (minimum night consumption) was measured by visiting the properties such as hospital, bus stand, etc., where the night consumption was observed and averaged out. Such LNF was established and was observed as 4.9, say 5 litres per connection per hour on an average basis, i.e., 0.083333 L/m. It may be observed that it is quite less compared to MNF. NRW values are computed by subtracting LNF from the values of MNF. The computation is shown in Table 12.4 and graph is shown in Figure 12.34.

12.18 Model Calibration and Validation

Model calibration is the process of adjusting model input data so that the simulated hydraulic quality output sufficiently mirrors observed field data. By generating an accurate set of basic inputs that provide a good picture of the real network and its components, the scope and difficulty of calibration is reduced.

Following calibration, model validation uses an independent field data set to check that the model is well calibrated. The calibrated model is run under settings different from those used for calibration in the validation step, and the results are compared to field data. If the model results closely approximate the field results for an appropriate time period, the calibrated model is considered as validated.

Calibration is a challenging task as a large numbers of input parameters are involved, such as nodal elevations and demands including water losses, pipe length, diameter, and roughness coefficients, the rate of water supply and water levels at sources, pump characteristics, and valve settings. Calibration depends on the accuracy of gathering information of these parameters from field. Therefore, it is desirable that some of these parameters are measured accurately during field observations so that there are few uncertain parameters requiring adjustments during calibration. Further, instead of developing different flow conditions to get the flow and pressure readings, an online flow and pressure measurement with data logger provides fairly accurate flow and pressure at various operating conditions in a day. Field observations and measurements provides reliable data regarding pipe length and material, ground elevations, operational settings and status of valve and water supply level at source node. The nodal demands are obtained from the consumer usage information, i.e., from billing data and NRW is adjusted. The corrosion and deposition processes over a time after the pipe has been installed make difficult to determine actual pipe diameter. Therefore, normal pipe diameter obtained during field observation is used in the hydraulic model and the roughness coefficient is adjusted to compensate the change in diameter. In general, following data obtained during field observations are considered fairly reliable and not adjusted during calibration: (1) Pipe length, diameter and material; (2) nodal elevations; (3) water levels in the reservoirs at start; (4) Valve settings, if any; and (4) pump operational schedule. However, the nodal demands and pipe roughness coefficients are less reliable and therefore may need adjustment during calibration.

Several methods have been suggested to simultaneously adjust pipe roughness coefficients and nodal demands (Bhave et al., 2006), or pipe roughness coefficient only with the assumption that nodal demands are fairly accurate and do not need any adjustment. Instead of judging the calibration through only differences and/or ratios of the observed to the predicted pressure or head loss differences between the test nodes and nearby boundary locations (reservoirs/tanks) having known pressures or heads (Walski 1986), it is preferable to judge the calibration through minimising the summation of the sum of square of difference between measured and simulated value of pressure heads at test nodes.

12.19 Interpretation Of Hydraulic Model Results

WDS models generate a large amount of output. The amount of calculated information increases with increasing model size and for EPS, the duration of the model run. Modern distribution system analysis software typically provides a range of graphical and tabular displays that help the user to go through the large amount of output data so that it may be efficiently analysed.

Using the hydraulic model L-section of the pipeline showing the graph of HGL and the distance can be plotted. This is very useful in transmission mains where the location of air valves and scour valves need to be determined.

12.20 Monitoring of Key Performance Indicators

The Government of India and the State Governments aim to provide drinking water supply in adequate quantity and of prescribed quality to every household through piped water supply with household tap connections and encourage ULBs to adopt 24×7 continuous water supply. 24×7 water supply is achieved when potable water is supplied for 24 hours in a day for 7 days in a week, in adequate quantity, with desired pressures, and assured quality at consumer’s locations. Ministry of Housing and Urban Affairs (MoHUA) has developed service level benchmarks for assessing performance of ULBs in providing water supply services. The key performance indictors (KPIs) given in following Table 12.5 are to be monitored and updated by the concerned ULBs/water authorities.

Table 12.5: Key Performance Indictors (KPIs)

S. No.	Key Performance Indicators (KPIs)	Targeted benchmark	Updated values for city/town by ULBs
1	Coverage of water supply connections (%)	100	
2	Per Capita Supply of Water (LPCD)	135	
3	Extent of Metering of Water Connections (%)	100	
4	Extent of NRW (%)	15	
5	Continuity of Water Supply (Hours)	24	

6	Quality of Water Supplied (%)	100	
7	Efficiency in Redressal of Customer complaints (%)	80	
8	Cost Recovery in Water Supply Services (%)	100	
9	Efficiency in Collection of Water Supply Related Charges (%)	90	

12.21 Strategy to Upgrade to Continuous System of Supply

Stages required for conversion to 24×7 can be summarised as under:

- Planning and design
- Actual Conversion to 24×7
- Long-term operational stage

The above strategy is summarised as under:

Apart from the technical measures, tariff strategy is required to save water by discontinuation of flat rates and charging on volumetric basis by adopting tariff on telescopic rate structure. Other measures such as organisational, commercial, policy, and budget are equally important. Summary of these strategic measures are shown in Figure 2.11, 2.12 & 2.13 in Chapter 2 of this manual.

All the above measures should be taken into consideration. If technical measures alone are taken but other measures are not, then the goal of conversion to 24×7 would not be achieved.

Activity Chart for Adoption of 24×7 Water Supply

Figure 12.35 shows the common activities which may be considered by ULBs/Water authority for adopting 24×7 water supply system.

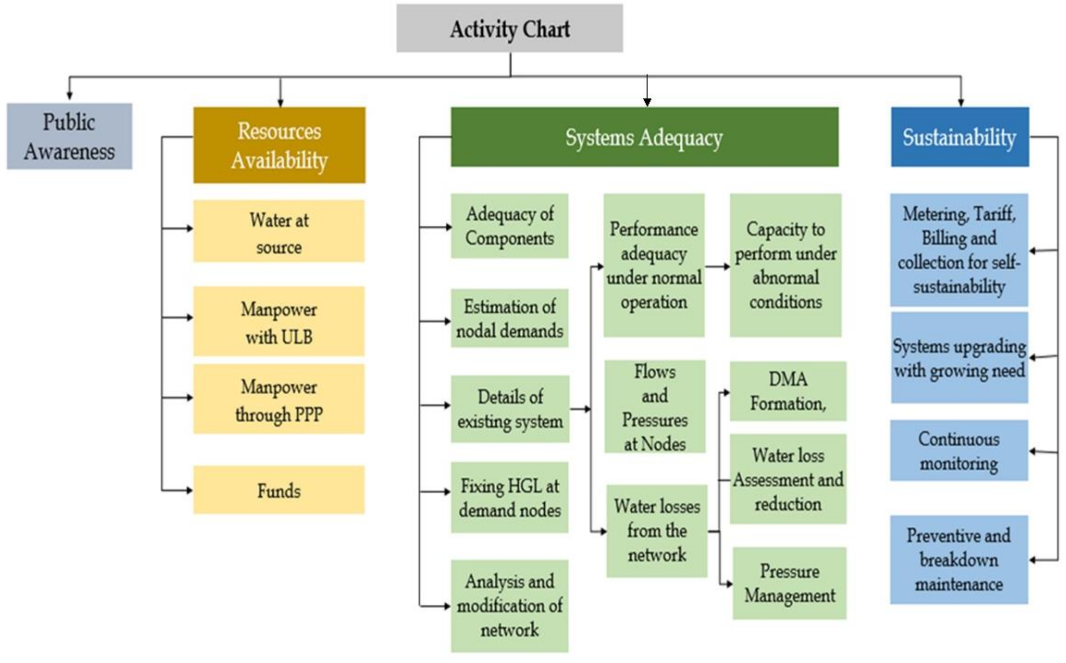


Figure 12.35: Activity Chart for Adoption of 24×7 Water Supply System

CHAPTER 13: WATER METERS

13.1 Introduction

A water meter is a scientific instrument for accurate measurement of quantity of water distributed to the consumer and efficient management of urban water supply. At present, about 20%–30% of these meters are non-functional due to poor selection, lack of maintenance and intermittent water supply.

Water meters play a very important role in transforming intermittent system to a 24×7 water supply system. 100% metering is required for demand management and water conservation. Without 100% metering, 24×7 pressurised water supply system is not possible. Using water meters, real-time data on the volume of water being consumed by a consumer (residential or bulk) can be obtained, thereby encouraging the consumer to use water more efficiently. Flowmeters are the instruments installed for the measurements of the volume of water consumed or discharged from the source of water like a river or bore well or dam/reservoir to the water treatment plant (WTP) and subsequently, to the district metering areas (DMA). Based on the requirement, some other features like pressure, temperature, turbidity, and water quality parameters can also be measured by the flowmeter.

Current problems with meters functioning in water supply systems include:

i. Intermittent water supply can cause fluctuations in water pressure, affecting the meter's accuracy. This can lead to overbilling or underbilling and damage to the meter itself. Intermittent supply can also lead to air in water lines, which will also affect accuracy of the meter as well.
ii. Water meters can become inaccurate over time due to wear and tear or damage, or the meter may not be properly sized, or air can get trapped in water meters. Foreign material in the water supply (scale, rust, or sand from main breaks) can also damage metering components.
iii. All meter types are susceptible to tampering, but mechanical meters are particularly more susceptible. In India, most meters used today are mechanical, which means they can be easily tampered with. Customers can remove or damage parts of the meter to reduce their water bills or avoid paying altogether.
iv. Some meters manufactured can be of poor quality, resulting in an inaccurate readings or frequent breakdowns. This can lead to overbilling or under billing and additional costs for repairs or replacement. Hence, the domestic water meter sizes varying from 15 mm to 50 mm shall conform with IS 779 and ISO 4064. Also, bulk water meter sizes ranging from 50 mm and above shall conform with IS 2373 and ISO 4064.
v. Meter installations can be susceptible to tampering and fraud. Some customers may attempt to tamper with their water meter or steal water by bypassing the meter.
vi. Water meters can malfunction for various reasons such as mechanical failure or electrical issues with meter reading endpoints which can result to inaccurate readings or a complete meter failure.
vii. Many water supply systems do not have adequate resources to maintain the meters. This can result in malfunctioning meters that do not provide accurate readings.

Water utilities can implement regular maintenance and repair programmes for water meters, take anti-tampering measures to prevent theft and misuse of water, and use modern and accurate metering technology. Additionally, efforts can be made to raise awareness among customers about the importance of paying for their water usage and the negative consequences of tampering with meters.

The challenge of regular maintenance and ensuring good quality meters can be overcome by adopting policy of procuring and maintaining the meters by water utility and recovering the cost from households. With this arrangement the consumers will have a sense of responsibility on the meters and will try to maintain the meter and meter boxes. By this, it is possible to ensure that the water meters function throughout the project life. Metering policy as discussed in Section 13.2 shall be referred.

Management of water resources in a system is a function of the measurement of quantity of water at source and its effective usage. They are indispensable for understanding the quantity of water being distributed in a system and its usage. Flowmeters are used to measure the quantity of water entering into the water supply systems from different sources such as water works, water treatment plants, or bulk water suppliers, and water meters are used to measure the quantity of water that is delivered to each metered consumer in the system.

Therefore, metering fulfils the need to know accurately the water produced and distributed by clear understanding of water balance. A well-implemented metering system in the water distribution network shall also assist technical staff in identifying the location where water loss/leakage is observed by comparing the water meter readings at the point of release of water with the readings at the consumer end. By estimating the level of water losses in a water supply system, unauthorised/illegal connections can also be identified.

The data obtained from a metering system also allows water managers to make a decision matrix for capital investments, maintenance, staffing, and various other aspects of the water supply systems. Therefore, water metering is an excellent application of the principle “to measure, is to know”. The knowledge of how much water is being used in the water distribution system is the key element in controlling the water loss and revenue loss thereof.

The water tariffs based on the quantity of consumption can be used for increasing the income of water supply agency, cross-subsidising needy consumers, and managing water consumption. However, a tariff policy cannot be implemented without a well-established metering system. Therefore, it is very essential in water supply system for installing a metering system in the cities/Urban Local Bodies (ULBs).

Water meter and flowmeters consist of four basic components: (i) a sensor to detect the flow, (ii) a transducer to transmit the flow signal, (iii) a counter to keep track of the total volume of water passed, and (iv) an indicator to display the meter reading.

The following points indicate how water meter is different from flowmeter:

- It is a quantity meter and not a flow rate meter.
- Water meter is a mechanical, or electromagnetic, or ultrasonic device whereas flowmeter may be mechanical or an electronic device.
- Water meter is always specified in two accuracies, i.e., lower range and upper range, whereas a flowmeter is specified in a single range accuracy.
- The upper range and lower range accuracies are 2% and 5% of the actual quantity, respectively, for the water meter, whereas it is variable for flowmeter, i.e., ±0.5% and ±5% as per the customer’s requirement.
- Importance is not given for repeatability and linearity in the case of water meter. whereas importance is given in the case of flowmeter because the accuracy of flowmeter performance is related to linearity and repeatability.

13.2 Metering Policy

Water metering means the method of measuring water consumption/use. In public water supply system water is supplied to the residential and commercial buildings, water meter measures the volume of water used by them. Water meter tracks water use. Besides monitoring water consumption, water meter saves the water. In intermittent system people store water and when fresh water is received, they throw the earlier stored water. Because of this, vicious cycle of decreased number of hours of supply, increase in number of zones and subzones is observed in practical operation of intermittent water supply scheme. With metering it saves such water loss.

Benefits of using water meters: The main benefits are as follows:

- Incentive to conserve water
- Volumetric pricing
- Detect water wastage.
- Estimating subsidies
- Regular supply
- Lower energy consumption

Therefore, in managing water supply, it is to be noted that the demand management is not possible without 100% metering.

13.2.1 State/ULBs Metering Policy

The State/ULB must frame and implement a clear metering policy for achieving 100% metering for reduction of NRW. The Metering Policy should have following basic principles embedded in it.

13.2.2 Legal Framework:

There should be a clause which provides “For calculating the amount payable by the owner for consumption of water supplied by the ULB/PHED/Board, the ULB/PHED/Board may determine the quantity consumed on the basis of reading recorded by a meter installed in the premises”.

13.2.3 Objectives of the Policy

The key objectives of the policy are:

a) To promote water conservation by encouraging efficient water use

b) To reduce Non-Revenue Water (NRW) and increase cost recovery.

c) To ensure fairness and equity to all Consumers in charging for water services.

d) To achieve 100% metering of all connections.

e) To set out roles and responsibilities of both ULB/PHED/Board and Consumer in relation to the installation of metered connections, maintenance of the water meters.

13.2.4 Scope of the Policy

This policy should cover types of consumers for metering, selection of meter, its specification and installation guidelines. This also covers the tariff, billing & collection and grievance redressal procedures related to metering. Further the responsibilities of the ULB/PHED/Board and consumers with regard to metering are described. Cost recovery and subsidy framework for urban poor may be covered under the policy.

The ULB/PHED/Board shall install the water consumption meters for all the consumers having water connections in a phased and progressive manner commencing with high water consumption and high revenue categories and all non-residential and new Consumers.

13.2.5 Ownership of meters

As per IS 2065: 1983 reaffirmed in 2022, consumer meters shall be owned by ULB/PHED/Board and fixed rental charges will be levied on the consumers along with the water charges. The rental charges shall be used for the specific purpose of routine and periodical maintenance of the meters and regular meter replacement program by the ULB/PHED/Board. The day-to-day safety and upkeep of the meters shall be the responsibility of the Consumer. If any wilful tampering or damage of the meters occurs either other than natural wear and tear, the ULB/PHED/Board will undertake replacement of such defective meter and the cost of such exceptional repair or replacement of meter shall be recovered from the consumer with advance notice.

Points to be considered while framing metering policy:

i. There is 100% consumer metering of all customers to realise differential water tariff on volumetric basis and ensure that all the meters are functional throughout the design period of 30 years.
ii. Each meter must be Geo tagged with GIS coordinates and shall be shown on GIS maps.
iii. Customer meter may be mechanical Class B double jet meter or AMR meters or Smart meters (electromagnetic or ultrasonic).
iv. For commercial, industrial and societies with high-rise buildings, automatic meter reading (AMR/ AMI) meters may be planned.
v. The meter shall be compatible with the communication technology.
vi. For procurement of meters through tender, ULB may empanel manufacturers of Mechanical, Ultrasonic and Electromagnetic meters conforming to latest IS or preferably ISO 4064_5: 2014, who possess certificates for performance, endurance and life cycle tests performed at Fluid Control Research Institute (FCRI) and Certified by FCRI. Only FCRI & OIML/ Measuring Institute Directorate (MID) tested and acceptable water meters shall be permitted / empanelled by ULB.
vii. The Standards for mechanical meters shall be as per ISO 4064_5: 2014/IS 2373:1991/IS 779:1994; electromagnetic as per ISO 4064_5: 2014 and OIML R49 and for ultrasonic meters the standard shall be ISO 4064_5: 2014 and OIML R49.
viii. Manufacturer must produce Life Cycle test certificate for at least one size of water meter each from domestic and bulk categories.
ix. Indian standard should be brought at par with International Organization for Standardization (ISO), till then meters should be as per ISO standard.
x. Utility should have their own repair workshop of customer meters, or the empanelled manufacturers shall have service centre in the city or at place in consultation with ULB.
xi. The consumers shall have all the options open to choose any type of water meter from empanelled makes and procure the same for installation on their connections.

xii. Cost of providing all meters till its warranty period shall be a part of project cost and ownership of meters shall rest with ULB. ULB to ensure that all the non-functional meters are replaced suitably after the warranty period. Some states like Odisha procure water meters and recover the cost from the households over a period of time to ensure proper repair and replacement. This model may be followed in other states.

xiii. ULB should engage meter provider/ manufacturer with O&M contract of meters for a period of 5 years so as to monitor the functioning of projects.

xiv. During this period the responsibility of proper maintaining meters with repair of meter shall be with meter provider.

xv. Mechanical meters should have warranty period of 5 years including repair & replacement of meters and if any repair work is required, the meter should be got repaired during the warranty period. The warranty period of Static meters may be considered as 10 years.

xvi. After warranty period non-functional meters shall be systematically got replaced

xvii. After warranty period if the non-functional meter is required to be replaced, consumers will be given 6-months' time for replacement, failing which they will be charged for 1.5 times more the consumption charged earlier and after 12 months the non-compliance will result in disconnection. Alternatively, the ULBs to procure and replace the meters and recover the cost in the water bills.

xviii. Bulk meters are installed at suitable places like head works, inlet and outlet of the WTP, inlet pipes of each service reservoir, entry of DMA, etc. so that NRW reduction becomes possible.

xix. All bulk meters are connected to the SCADA system or IoT systems.

xx. The test certificates by manufacturers shall be accepted for all the diameters for new installation. Only in case of disputed readings, the meter will be tested at Municipal facility.

xxi. The Ultrasonic / Electromagnetic Meters being accurate any variation in reading beyond ± 10% of previously recorded consumption will be considered erroneous and subject to meter scrutiny and site investigation.

xxii. Effective metering needs efficient reading and billing mechanism. Meter reading is encouraged using various metering and communication technologies as discussed.

xxiii. ULB will continue to play its role as a facilitator and regulator of water supply being an obligatory duty.

xxiv. The Repair / Maintenance / Testing facilities shall be locally made available by the manufacturers at reasonable rates. This will be one of the criteria while empanelling the manufacturers.

13.3 Sizing of Water Meters

The nominal sizes of domestic water meter ranges from 15 mm to 50 mm as per {IS 779: 1994 (Reaffirmed 2015)} and bulk water meter ranges from 50 mm and above as per {IS 2373: 1981 (Reaffirmed 2017)}. Sizing of water meter is done keeping in view the guidelines given in Indian standard {IS 2401: 1973} and {ISO 4064 Part II: 2014}.

In general, the main considerations are as follows:

i. Water meter should be selected according to the flow to be measured and not necessarily to suit a certain size of water main.

ii. The maximum flow should not exceed the maximum flow rating.

iii. The nominal flow should not be greater than the nominal flow rating.

iv. The minimum flow measured should be within the minimum starting flow of the meter.

v. Low head loss, long operating flow range, less bulky, and robust meter should be preferred.

13.4 Classification of Water Meters

Water meters are generally classified based on the different mechanisms used by the water meter to measure the flow of water passing through it. There are several types of flowmeter technology available in the market for bulk flow measurement and domestic water metering. The most prominent and widely used flowmeter for bulk flow measurement is the full bore, inline electromagnetic flowmeter. Traditionally, for household connections, mechanical Class B multijet meters were used. However, with advancements in technology, progressive water boards are opting for new smart technologies such as electromagnetic/ultrasonic static meters with automatic meter reading (AMR)/Advance metering infrastructure (AMI) solution, especially for high consumption customers. This is due to the various advantages they offer, such as the reduction of cycle time for billing and better customer service.

In India, currently, meters are broadly categorised into (i) Bulk flow and (ii) Domestic/Consumer revenue meters based on usage, as shown in Figure 13.1.

Figure 13.1: Type of meters in India based on consumer category

Water meters can also be classified based on metrological characteristics, as given in ISO 4064-3 Standard. The water meter classification is divided according to the water meter transitional flow rate (Q2) and the minimum flow rate (Q1), which indicates the sensitivity of the water meter, as shown in Table 13.2 below. The old regulations classify water meters into four classes: Class A, Class B, Class C, and Class D. Notably, Class D is the highest level as shown in Table 13.1.

Table 13.1: Water meter Classification- According to ISO 4064-3 (1999) Standard

Nominal Diameter	Qn (m3/hr)	Qmax (m3/hr)	Class A	Class B	Class C	Class D

			Qmin (litres/hr)	Qt (litres/hr)	Ratio	Qmin (litres/hr)	Qt (litres/hr)	Ratio	Qmin (litres/hr)	Qt (litres/hr)	Ratio	Qmin (litres/hr)	Qt (litres/hr)	Ratio
DN15	1	2	–	–		–	–		–	–		7.5	11.5	133
DN15	1.5	3	60	150	25	30	120	50	15	22.5	100	11.25	17.25	133
DN20	2.5	5	100	250	25	50	200	50	25	37.5	100	18.75	28.75	133
DN25	3.5	7	140	350	25	70	280	50	35	52.5	100	26.25	40.25	133
DN25	6	12	240	600	25	120	480	50	60	90	100	–	–	
DN32	6	12	240	600	25	120	480	50	60	90	100	–	–	
DN40	10	20	400	1000	25	200	800	50	100	150	100	–	–	

At present, Class A water meter has basically been withdrawn from the mainstream market, and Class B water meter is the most commonly used water meter in the market. Class C water meter has been more popular in the market because of its high measurement accuracy and ability to read at low flow. Class D water meter has higher requirements for manufacturing process and equipment and, at present, mainly offered by an ultrasonic water meter.

The water meter classification according to ISO 4064-3 Standard is shown in Table 13.2 below.

Table 13.2: Water meter Classification- According to ISO 4064-3 (2014) Standard

Size		Ratio	Q_4 Overload flow	Q_3 Nominal Flow	Q_2 transitional flow	Q_1 Min flow	Min reading	Max reading
DN (mm)	Inch		m³/hr		L/h		m³	
15	1/2"	80	3.125	2.5	50.000	31.250	0.001	999999
		160			25.000	15.625		
		250			16.000	10.000		
		400			10.000	6.250		
		500			8.000	5.000		
		630			6.35	3.968		
		800			5.000	3.125		
20	3/4"	80	5.00	4.00	80.000	50.000	0.001	999999
		160			40.000	25.000		
		250			25.600	16.000		
		400			16.000	10.000		
		500			12.800	8.000		
		630			10.16	6.349		
		800			8.000	5.000		
25	1"	80	7.875	6.3	126.000	78.750	0.001	999999
		160			63.000	39.375		
		250			40.320	25.200		

Size		Ratio	Q_4 Overload flow	Q_3 Nominal Flow	Q_2 transitional flow	Q_1 Min flow	Min reading	Max reading
DN (mm)	Inch		m³/hr		L/h		m³	
		400			25.200	15.750		
		500			20.160	12.600		
		630			16.00	10.000		
		800			12.600	7.875		
32	1 1/4"	80	12.5	10	200.000	125.000	0.001	999999
		160			100.000	62.500		
		250			64.000	40.000		
		400			40.000	25.000		
		500			32.000	20.000		
		630			25.40	15.873		
		800			20.000	12.500		
40	1 1/2"	80	20	16	320.000	200.000	0.001	999999
		160			160.000	100.000		
		250			102.400	64.000		
		400			64.000	40.000		
		500			51.200	32.000		
		630			40.63	25.397		
		800			32.000	20.000		
50	2"	80	25	20	400.000	250.000	0.001	999999
		160			200.000	125.000		
		250			128.000	80.000		
		400			80.000	50.000		
		500			64.000	40.000		
		630			50.79	31.746		
		800			40.000	25.000		

Q1 - lowest flow rate; Q2 - transitional flow rate; Q3 - permanent flow or nominal flow rate; Q4 - highest flow rate

13.5 Detailed Description of Meters and Applications

Mechanical meters have moving parts that detect the flow, such as a piston or impeller. They make up the vast majority of meters used in water distribution systems especially to measure the consumption and billing purpose at the domestic level. Electromagnetic and ultrasonic meters have no moving parts but detect the flow through the meter using electromagnetic waves and ultrasound waves, respectively. They are mostly for bulk metering, such as in very large pipes and/or where a high accuracy metering is required like DMA measurement.

Mechanical water meter like the single jet, multijet, piston type, and electromagnetic and ultrasonic meters are used for domestic purposes. The preferred diameter size for domestic metering is 15 mm to 40 mm. Bulk water meters are used to measure high water consumption for billing/water audit purposes by bulk consumers like commercial complexes, industries, etc. Generally, Woltman water meters (mechanical type), electromagnetic and ultrasonic water meter are used for bulk metering. The preferred diameter size for bulk metering is 50 mm to 150 mm.

As described in clause 8.5.8 of IS 17482: 2020 Standard, for water audit application, meters shall be preferred both at domestic metering and DMA. Sub-classification of different meters is shown in Figure 13.2 as under:

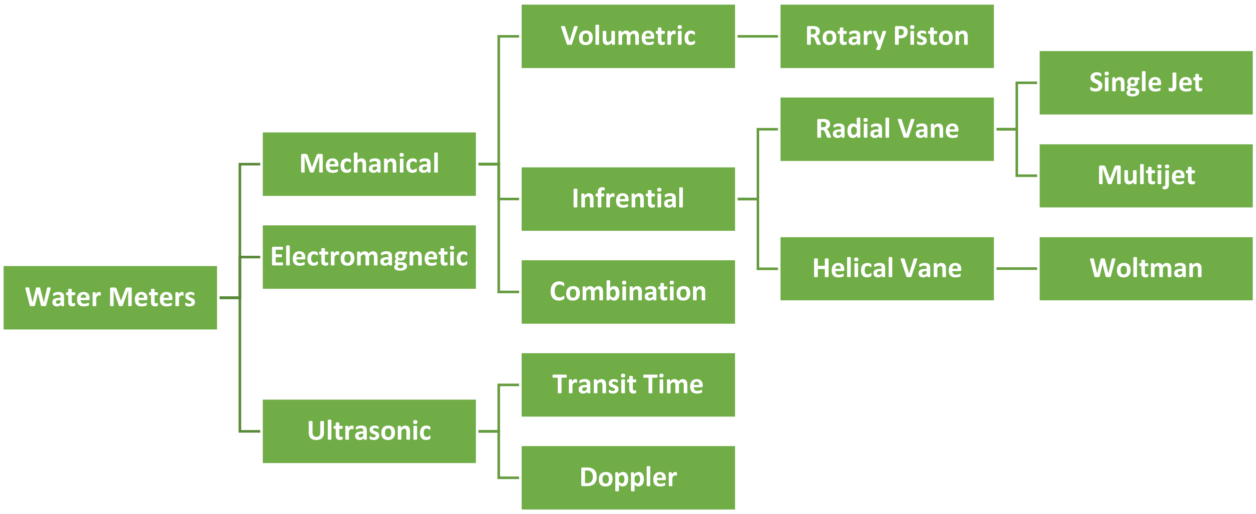


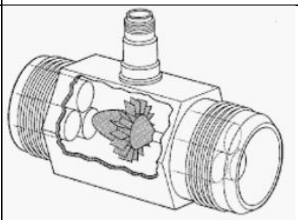
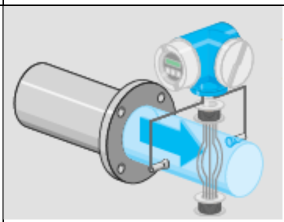
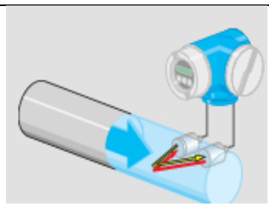
Figure 13.2: Sub-classification of different meters

Comparison of mechanical, electromagnetic and ultrasonic meters is given in Table 13.3 below.

Table 13.3: Comparison of Mechanical, Electromagnetic and Ultrasonic Meters

S. No	Attributes	Mechanical meters	Electromagnetic meters	Ultrasonic meters
1.	**Working principle**	Paddle Wheel, Turbine or mechanical with moving counter	Electromagnetic induction principle Faraday's Law measurement	Ultrasonic Measurement principle Time of Flight measurement
2.	**Build**	(i) Moving parts are present (ii) Mechanically and magnetically coupled meter (iii) Dry/Wet Dial meter	(i) No moving parts (ii) Sensor in-build (iii) Dry Dial meter (iv) Can be inline type threaded/flanged end	(i) No moving parts (ii) Sensor in-build (iii) Dry Dial meter (iv) Can be inline type threaded/flanged end
3.	**Available sizes**	15 mm – 500 mm	15 mm – 3000 mm	15 mm – 4000 mm
4.	**Application**	Domestic meter: 15 mm to 40 mm Bulk meter (Woltman): 50 mm to 500 mm or more	Domestic meter: 15 mm to 40 mm Bulk meter: 50 mm to 3000 mm or more	Domestic meter: 15 mm to 40 mm Bulk meter: 50 mm to 4000 mm or more
5.	**Standards**	ISO 4064/IS 2373/IS 779	ISO 4064 and OIML R49	ISO 4064 and OIML R49
6.	**Water quality**	Suitable for potable water. Highly critical with suspended particles as it clogs the moving parts.	Suitable for potable water. No impact of particles.	Suitable for potable water. Highly critical with suspended impurity and turbidity as it deposits on the sensor face.
7.	**Accuracy**	Better than ±2% for upper flow range and ±5% for lower flow range.	Better than ±2% for upper flow range and ±5% for lower flow range.	Better than ±2% for upper flow range and ±5% for lower flow range.
8.	**Installation orientation**	Good performance under horizontal installation.	Good performance under horizontal, vertical, and inclined installation.	Good performance under horizontal, vertical, and inclined installation.
9.	**Periodic maintenance**	Very high as it has a lot of moving parts and wear and tear is a regular issue. There is no indication of wear and tear as no warning available other than the high-pressure drop.	Less and the expected life of the meter is a minimum of 10 years.	Less and the expected life of the meter is a minimum of 10 years.
10	**IP-68**	Yes, with copper can	Yes	Yes

S. No	Attributes	Mechanical meters	Electromagnetic meters	Ultrasonic meters
	availability	register version.		
11	**Operating accuracy at low flow**	Poor	Good	Good
12	**Cost**	Low initial cost but the high cost of maintenance due to moving parts and more frequent replacements like jamming of rotating wheels, counters, etc.	Cost increases with diameter.	Moderate.
13	**Advantages**	(i) Suitable for higher flows (ii) Can sustain hostile flow conditions (iii) External and internal regulator facilitates easy calibration (iv) Robust construction (v) Easy Maintenance	(i) Suitable for wide range of flows (very low to high flows) (ii) Less sensitive to flow disturbances. (iii) Ready for Automatic Meter reading for water SCADA compliant (iv) Do not measure the air in the pipe (v) Can be installed in any Orientation. (vi) Life of meter 10 to 15 yrs. (vii) Low Maintenance required (viii) U0D0 – Meter works accurately with zero upstream and downstream length for installation	(i) Suitable for wide range of flows (low to high flows) (ii) Less sensitive to flow disturbances. (iii) Ready for Automatic Meter reading for water SCADA compliant (iv) Do not measure the air in the pipe (v) Can be installed in any Orientation. (vi) Life of meter 10 to 15 yrs. (vii) Low Maintenance required. (viii) U0D0 – Meter works accurately with zero upstream and downstream length for installation

S. No	Attributes	Mechanical meters	Electromagnetic meters	Ultrasonic meters
14	**Disadvantages**	(i) Less sensitive to low flow (ii) Approach conditioning piping is required (iii) Limited to higher flows (iv) Bush leak problems (v) Meters measures air (vi) Brass body meter – prone to theft	(i) Costlier than mechanical meters (ii) Water must be free from solid dirt particles	(i) Costlier than mechanical meters (ii) Water must be free from solid dirt particles. (iii) Ultrasonic sensors may need to be cleaned periodically.
15	**Representation**			

13.6 Mechanical Meters

Mechanical meters are further classified in three categories, i.e., volumetric, inferential, and combination meters.

13.6.1 Volumetric Meters

Volumetric meters directly measure the volume of flow passing through them. Most volumetric meters use a rotating disk to measure the flow and are known as rotating piston meters. For application of volumetric meters, the total dissolved solids (TDS) level in water should be lower than 200 ppm.

Rotary Piston Meters

Rotary piston meters are positive displacement meters that use a rotating cylindrical piston to measure 'packets' of water moving from the inlet to the outlet of the meter.

Positive displacement meters are popular for their accuracy, long life, and moderate cost and are used for most domestic applications. Rotating piston meters are sensitive to sand and/or other suspended solids in the water that can get clogged between the piston and chamber wall, thereby clogging the meter. These meters are also sensitive to low flows and are particularly suitable for applications where the water flow rates are low or where frequent onsite leakage occurs. Rotary piston metre is shown in Figure 13.3.

The main disadvantages of rotating piston meters are:

(i) being sensitive to suspended solids in the water;
(ii) prone to relatively high-pressure losses; and

Figure 13.3: Rotary Piston Meters

(iii) bulky and expensive than other meter types.

13.6.2 Inferential Meters

Inferential meters do not measure the volume of water passing through them directly but infer the volumetric flow rate from the velocity of the water. Two categories of inferential meters commonly used are:
(i) meters using a radial vane impeller; and
(ii) meters using a helical vane impeller.

Radial vane impeller meters are further classified into a single jet and multijet (also known as multiple jets) meters. Helical vane impeller meters are also called Woltman meters and use a propeller-like vane to increase the water velocity. Multijet meters are widely accepted in countries such as Brazil, Malaysia, Indonesia, India, Vietnam, etc., where the water supply system is intermittent.

13.6.2.1 Single Jet Meters

Single jet meters are inferential meters consisting of an impeller with radial vanes (also called a fan wheel) and use a single flow stream or jet to move the sensor. The rotational speed of the impeller is converted into a flow rate, which is registered on the meter. It is critical to precisely control the path of water through the single jet meter to obtain accurate readings. Thus, the inside portion of the single jet meter has to be manufactured to strict tolerances. Single jet water meter and its cross-sectional view is shown below in Figure 13.4.

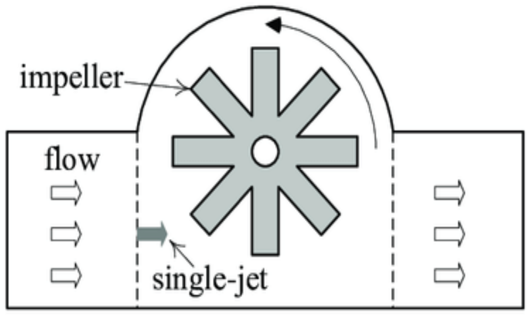


Figure 13.4: Single jet water meter and its cross-sectional view

The accuracy of single jet meters reduces due to wear in the moving parts with continuous usage over a period of time. In particular, the starting flow and accuracy of metering at low flow rates may deteriorate, and thus, older meters tend to under-register at low flow rates. At higher flow rates, the error can be positive or negative and may be exacerbated by sediments or deposits accumulating inside the meter. Air moving through the meter will also be registered as water, and thus, can lead to over-register of water flow. Traditionally, the metering chamber is made out of brass, but plastics are also becoming popular. The composite body (engineered plastics) for water meters makes it economical. Brass chambers make the single jet meter expensive, especially in larger diameters. Single jet meters are thus mostly used in the size range of 15 mm to 40 mm.

Advantages of brass chambers for water meters:

i. Steady water meter;
ii. Protects the register can inside;
iii. No health hazards;
iv. Scrapped and re-use;
v. End connection installation.

Disadvantages of brass chambers for water meters:

A. Makes water meter heavier;
B. Increase the cost of manufacturing by 5%–6%;
C. Prone to theft due to the high value of scrap;
D. Drift in accuracy over the period of time;
E. Installation is restricted to horizontal position only.

13.6.2.2 MultiJet Meters

MultiJet meters are inferential water meters that use an impeller with radial vanes. The operation of MultiJet meters is similar to that of single jet meters, except that MultiJet meters use several jets to drive the impeller at multiple points. This implies that the forces applied on the impeller are better balanced than in single jet meters, thereby reducing wear on the moving parts and provides greater durability.

They are similar in construction to that of single jet meters, although MultiJet meters tend to be slightly larger in overall size. MultiJet meters are fitted with removable strainers on the inlet side of the meter, to facilitate the cleaning of the same. A second internal strainer often covers the openings of the metering chamber. The internal strainer, if clogged, can affect the accuracy of the meter, thereby causing over-registration of the flow. MultiJet water meter and its cross-sectional view is shown in Figure 13.5.

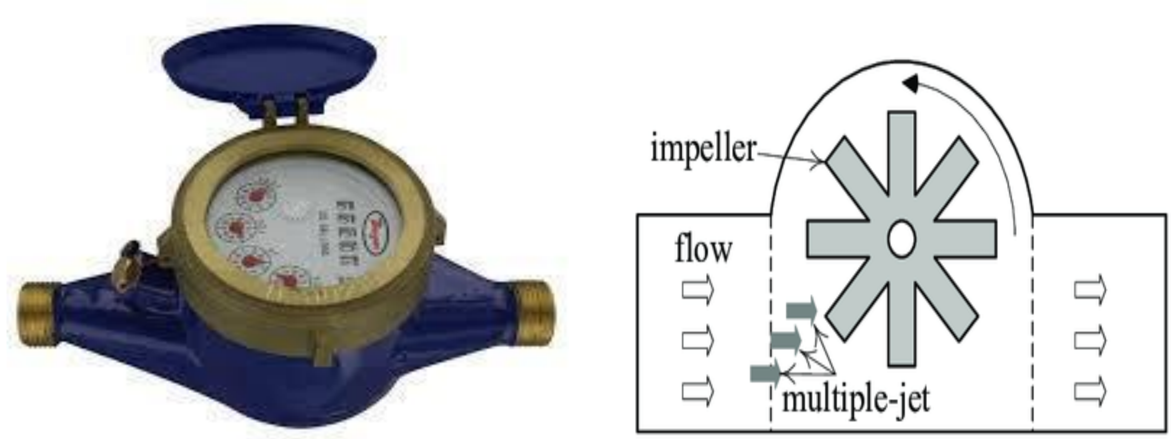


Figure 13.5: MultiJet water meter and its cross-sectional view

They normally use an internal bypass with a regulating screw to adjust the flow passing through the impeller. This allows the manufacturer to adjust the meter's error curve to achieve the best accuracy before sealing the meter to prevent meter tampering. Traditionally, the meter body is made out of brass, but plastics are also becoming popular. The composite body (engineered plastics) for water meters makes it economical.

MultiJet meters use reliable and tested metering technology and normally have longer lifespan due to the balanced forces on the impeller. They are not sensitive to the velocity profile in the pipe and are tolerant of small, suspended solids in the water.

The disadvantages of MultiJet meters are:

A. Sensitivity to the installation position, thereby affecting accuracy;
B. Often bulkier than single jet meters;
C. Not being sensitive to low flow rates;
D. Starting flow rate can deteriorate significantly with time;
E. Accuracy may be significantly affected by clogs in jet openings, if any;
F. A brass body prone to theft, due to the high value of scrap;
G. Meter life of three to four years.

13.6.2.3 Woltman Meter

The Woltman meter is an inferential meter that uses an impeller with helical vanes, which resembles a fan or boat's propeller. As water flows over the helical vanes, it causes the impeller to rotate, and the rotation is then transmitted to the dial via reduction gearing.

There are two different types of Woltman meters, horizontal turbine, and vertical turbine. Horizontal Woltman meters have their inlets and outlets directly in line with the pipeline, and the axle of the helical vane is parallel to the flow. Water flows directly through the meter with minimal disturbances by the meter body. Horizontal Woltman meters are used in a large range of pipe sizes, typically having a diameter between 40 mm and 600 mm. Vertical Woltman water meter is designed for industrial and irrigation applications in sizes 50 mm and 200 mm for the cold meter. Both vertical turbine and horizontal turbine are shown in the Figure 13.6 and Figure 13.7, respectively.

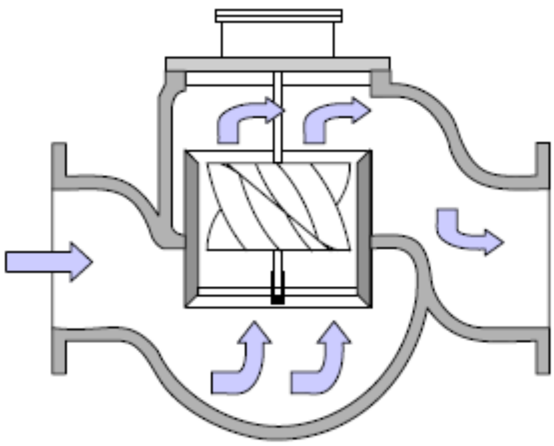


Figure 13.6: Vertical Turbine

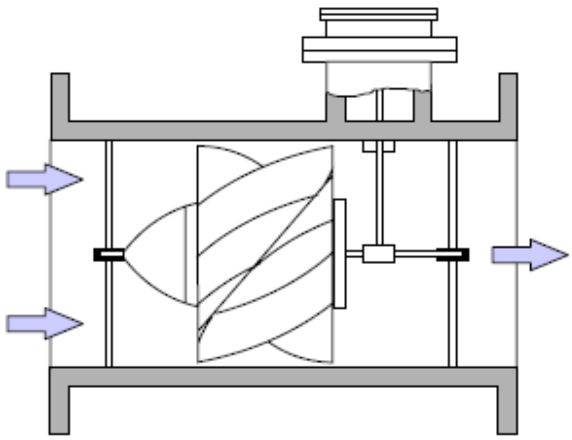


Figure 13.7: Horizontal Turbine

Woltman meters are affected by flow distortions or changes in meter dimensions that may interfere with the way water passes through the meter. Deposits in the meter can cause over-registration at medium flows and under-registration at low flows.

All Woltman meters have dry, sealed dials. The easy passage of water through horizontal Woltman meters reduces pressure loss through the meter. However, since the transducer needs to turn the circular movement of the impeller through 90 degrees to connect it to the counter, greater torque is required, which reduces the meter's sensitivity to low flows. Another limitation of Woltman meters is that they are sensitive to disturbances in the flow passing through them. Bends or valves close to a horizontal Woltman meter can affect the meter's accuracy. Spiralling flow, caused by two successive bends in different planes, is particularly unfavourable for their accuracy. Woltman meter is shown in Figure 13.8.

Figure 13.8: Woltman Meter

13.6.3 Combination Meters

They do not use a specific mechanism to measure the flow but are made up of two meters of different diameters that are combined to measure a wide range of flow.

They are generally used to measure high flow rates with extremely widespread flow profiles and also to measure very small flow rates for leakage detection and are ideal for fire service pipes. Combination Meter is shown in Figure 13.9.

Figure 13.9: Combination Meter

13.7 Electromagnetic Water Meters

Electromagnetic water meters function on the principle of electromagnetism, called Faraday's Induction Law, to measure the velocity of the water passing through it. In an electromagnetic meter, a magnetic field is created across the pipe. When water, which is an electrical conductor, moves through the magnetic field, a voltage is induced which is detected by electrodes in the body of the meter. The voltage is directly proportional to the flow velocity, which is used to calculate the volume. Domestic electromagnetic meter DN15 to DN40 is shown in Figure 13.10 and bulk electromagnetic meter DN50 to DN300 is shown in Figure 13.11.

Figure 13.10: Domestic Electromagnetic meter DN15 to DN40

Figure 13.11: Bulk Electromagnetic meter DN50 to DN300

The voltage is measured by two electrodes placed at right angles to the magnetic field. The sensor measurement is transmitted via an electric signal to an electronic counter, which converts the velocity readings to volume. The volume of consumption and/or flow rate is normally displayed on an LCD screen.

Electromagnetic water meters are accurate within their measuring range. Electromagnetic water meter's accuracy is defined as +/-5% for lower operating flow range and +/-2% for upper operating flow range (as per ISO 4064 Standard).

Advantages of electromagnetic meters are as follows:

i. No obstruction to flow;
ii. No pressure loss;
iii. No moving parts subject to wear, therefore there is hardly any maintenance;
iv. Highly accurate and immune to variations in fluid density, pressure, viscosity, or temperature;
v. Measures only water, no air;
vi. No drift in accuracy over the product life;
vii. Composite body, not prone to theft;
viii. Very good low flow; and
ix. Meter life more than 10 years according to R800 metrology.

The electromagnetic domestic meters operate on battery and battery life is minimum of 10 years.

13.8 Ultrasonic Water Meters

Ultrasonic water meters utilise the properties and behaviour of sound waves passing through moving water. The ultrasonic water meters are designed on the transit time principle.

A transit time ultrasonic meter has two sound transducers mounted at opposite sides of the pipe at an angle to the flow. Each of these sound transducers will, in turn, transmit an ultrasound signal to the other transducer. The differences in the transit times of the signals determine the velocity and the volume is derived from the velocity. The measuring principle is called 'bidirectional ultrasound technique based on the transit time method', which is a proven, long-term stable and accurate measuring principle.

Domestic ultrasonic meter DN15 to DN40 is shown in Figure 13.12 and bulk ultrasonic meter

DN50 to DN300 is shown in Figure 13.13.

Figure 13.12: Domestic Ultrasonic Meter DN15 to DN40

Figure 13.13: Bulk Ultrasonic Meter DN50 to DN300

The accuracy of the transit time ultrasonic meters depends on the ability of the meter to accurately measure the time taken by the ultrasound signal to travel between the sound transducers. Larger pipes have longer path lengths and thus, the speed of the signal, and the flow rate can be measured with higher accuracy. Transit time meters work better in clean fluids and thus, are ideal for drinking water pipes. They measure the average velocity of fluid but are sensitive to the velocity profile in a pipe. In some cases, multi-beam devices are used to improve meter accuracy.

The ultrasonic water meter's accuracy is defined as +/-5% for lower operating flow range and +/-2% for upper operating flow range as per ISO 4064_5:2014 Standard.

Deposits on the inside pipe surface can affect signal strength and performance. The disadvantage of the ultrasonic water meters is that air in the liquid, turbulence, deposits on the sensors, and water hammer (pressure transients) affects its performance. Besides, the ultrasonic water meters operate on battery and battery life is minimum of 10 years.

13.9 Installation and Testing of Water Meters

13.9.1 Installation of Water Meters

In order to ensure proper working of the meters, BIS has given guidelines in IS-2401:1973 and ISO 4064_5:2014 Part 5 for their installation, as per the drawing given in it. At the same time, following guidelines should be borne in mind while installing the meters.

i. The water meter being a delicate instrument shall be handled with great care. Rough handling including jerks or fall is likely to damage it and affects its accuracy.
ii. The meter shall be installed at a spot where it is readily accessible. To avoid damages and overrun of the meter due to intermittent water supply system, it is always advisable to install the meter so that its top is below the level of the communication pipes. This ensures that meter always contains water when there is no supply in the line. Also, the minimum straight length of pipe before and after the meter, as per the drawing, shall be observed. This assures laminar flow of water to the meter.
iii. The meter shall preferably be housed in a chamber with a protective lid. It should never be buried underground, installed in the open, or under a water tap so that water may not directly fall on the meter. It should be installed inside inspection pits built out of bricks or

concrete and covered with the lid. It should not be suspended.

iv. The meter shall be installed so that the longitudinal axis is horizontal, and the flow of water should be in the direction shown by the arrow cast on the body.

v. Before connecting the meter to the water pipe, it should be thoroughly cleaned by installing in the place of the water meter a pipe of suitable length and diameter and letting the passage of a fair amount of water flow through the pipework to avoid the formation of air pockets. It is advisable that the level of the pipeline where the meter is proposed to be installed should be checked by a spirit level.

vi. Before fitting the meter to the pipeline, check the unions nuts in the tail pieces and then insert the washers. Thereafter, screw the tail pieces on the pipes and install the meter in-between the nuts by screwing. To avoid its rotation during the operation, the meter should be kept fixed with suitable non-metallic clamps. Care should be taken that the washer does not obstruct the inlet and outlet flow of water.

vii. The protective lid should normally be kept closed and should be opened only for reading the dial.

viii. The meter shall not run with free discharge to the atmosphere. Some resistance should be given in the downside of the meter if static pressure on the main exceeds 10 m head.

ix. A meter shall be located where it is not liable to get the severe shock of water hammer which might break the system of the meter.

x. Owing to the fine clearance in the working parts of the meters, they are not suitable for measuring water containing sand or similar foreign matter, and in such cases, a filter or dirt box of the adequate effective area shall be fitted on the upstream side of the meter. It should be noted that the normal strainer fitted inside a meter is not a filter and does not prevent the entry of small particles, such as sand.

xi. Where intermittent supply is likely to be encountered, install an appropriate air valve before the meter to reduce inaccuracies and to protect the meter from being damaged. At higher altitude, if the meter is installed as above, the problem will be eliminated.

xii. Every user expects a problem-free installation of the meter and, thereafter, only accurate reading. Regular monitoring is desirable in order to avoid failures.

xiii. The meter is installed in the pipeline using flanged or threaded connections giving due consideration for conditioning sections. It should be seen that stress-free installation is carried out in the pipeline.

xiv. Installation in ‘U’ shape is essential for intermittent water supply to avoid direct air pressure on impeller during starting of supply hours, which may damage impeller.

13.9.2 Testing and Calibration of Water Meters

The testing and calibration of a water meter is essential before putting it into use as it is a statutory requirement. It is also essential to test it periodically in order to ascertain its performance during the life of the meter as its accuracy of measurement may deteriorate beyond acceptable limits over a period of time. Calibration consists of comparing the meter reading with the reading obtained from a standard of higher accuracy than the test meter and with established uncertainty. Meters may also be tested using volumetric testing using calibrated tanks. Calibration is usually done in a laboratory at a variety of flow rates, as well as varied densities and temperatures. The calibration factors of the meter are determined during calibration.

The accuracy of water meter is divided into two zones, i.e., (i) lower measurable limit in which ±5% accuracy from minimum flow Q1 to transitional flow Q2 (exclusive), and (ii) upper measurable limit in which ±2% accuracy from transitional flow Q2 (inclusive) to maximum flow Q4 (as per ISO 4064_5:2014 Standard).

The metering accuracy testing is carried out at as per {IS 779: 1994/ {ISO 4064_5:2014} Q_{min}/Q1, Q_t/Q2, and Q_n/Q3 separately.

Where

Q_1: Minimum flow rate at which the meter is required to give indication within the maximum permissible error tolerance. It is as mentioned in IS 779:1994 and is determined in terms of numerical value of meter designation in case of ISO 4064_5:2014. Q1 value is derived from the ratio, i.e., Q1 = Q3/ratio.

Q2: The transitional flow rate at which the maximum permissible error of the water meter changes in value. Q2 is 1.6 times of Q1.

Q3: Permanent flow rate as mentioned in ISO 4064-2014 for each size of the meter.

Q4: The overload flow rate at which the meter is required to operate in a satisfactory manner for short periods of time without deterioration. Q4 is 1.25 times of Q3.

Dynamic Ratio – Q3/Q1

13.9.2.1 Procedure for Conducting the Test

Water meter is fixed on a test bench horizontally or vertically, or in any other position for which it is designed and with the direction of flow as indicated by the arrow on its body. By adjusting the position of the regulating valve on the upstream side, the rate of flow is adjusted. At the desired rate of flow, the difference in pressure gauge readings fitted on the upstream and downstream side of the water meter is noted. The flow is now stopped with regulating valve and the measuring chamber is emptied and zero water levels on the manometer attached to the measuring chamber are correctly adjusted. The initial reading of the water meter from its recording dial is noted. Now the flow at the set rate is passed through the water meter and the discharge is collected in the measuring chamber. After passing the desired quantity of water through the meter, the flow is once again stopped. The discharge, as recorded by the measuring chamber, is noted. The final reading of the water meter is noted. The difference between the initial and final readings of water meter provides the discharge figure recorded by the water meter. Now the discharge recorded by measuring tank is treated as ideal. The discharge recorded by water meter is compared with this ideal discharge. If the quantity recorded by water meter is more than the ideal, the meter is called running fast or vice versa. The difference in the quantity recorded by the meter from the ideal quantity is considered as an error. This error is expressed in percentage.

If the limits of error for the meter exceed as specified in the IS concerned, the meter is readjusted by the regulator if it is available in the meter. A change in position of the regulating screw will displace the error curve (calibration curve) in parallel to the former position. With the closing of the regulating orifice, the curve will shift upward while opening the same will lower the curve. If the curve does not get into an acceptable limit, the meter is not used. Some of the organisations are accepting accuracy limit for repaired water meter that is double the value of new water meters at respective zones, i.e., for upper zone accuracy is ±4% and for lower zone accuracy is ±10%.

The meter testing is normally carried in the meter testing laboratory with the help of one of the following methods.

1. Gravimetric
2. Volumetric
3. Prover
4. Master or reference meter
5. Tow tank – current meter calibration

13.9.2.2 Point Calibration Test

All the meters manufactured shall be tested/calibrated for accuracy at Q1, Q2, and Q3 flow rates and shall meet the acceptance criteria of errors less than the MPE, as per ISO 4064 standard.

Calibration consists of comparing the meter reading with the reading obtained from a standard of higher accuracy than the test meter and with established uncertainty. The standard may be a reference master meter or a complete test bench which are traceable back to more fundamental measures of mass, time, and volume. Test report format is as under:

Meter Serial Number	3 Point Accuracy Test			Date of testing
	Q1 (%)	Q2 (%)	Q3 (%)	

13.9.2.3 Lot Acceptance Test: Meter Testing from first lot of meters

Sample meters shall be sent to approved laboratory from the first lot of meters based on the sampling plan given at Table 13.4 for conducting the below tests.

i. Accuracy test at Q1, Q2, 0.35(Q2 +Q3), 0.7(Q2 + Q3), Q3 and Q4 as per ISO 4064 standard
ii. Pressure loss test
iii. Static pressure test
iv. AMR communication test with the meter
v. IP68 test on two sample meters
vi. Life cycle test on three sample meters

Table 13.4: IS 779 Sampling plan for Sample Size and Criteria for Acceptance

Size of the lot	**Size of First Sample**	**Acceptance Number**	**Rejection Number**	**Size of Second Sample**	**Size of Cumulative Sample**	**Cumulative Acceptance Number**
Up to 50	5	0	1	-	-	-
51–150	13	0	2	13	26	1
151–280	20	0	3	20	40	3
281–500	32	1	3	32	64	4
501–1200	50	2	5	50	100	6
1201–3200	80	3	6	80	160	9
3201–10000	125	5	9	125	250	12
10001–35000	200	7	11	200	400	18

35001 and over	315	11	16	315	630	26

13.9.2.4 Certificates to be provided with the meters during QAP Approval

All the below certificates shall be submitted during QAP approval process.

i. OIML R 49/ MID (Module B+D or Module H1) Certificate
ii. MAP and the endurance certificate from approved laboratory for each size
iii. IP-68 Certificate from FCRI for the model of meters
iv. Certificate of Approval of Model as per Legal Metrology Act
v. ISO 17025:2005 certificate

13.9.2.5 Setting up a Test facility

The utility should set up a test facility in their premises to test accuracy, pressure loss, and static pressure of water meters to attend customer grievance.

13.10 Repairs, Maintenance and Troubleshooting of Water Meters

13.10.1 Introduction

The water meters are mechanical devices which normally deteriorate in performance over time. The fact that a meter does not show outward signs of any damage and has a register that appears to be turning does not mean that the meter is performing in a satisfactory way. It is necessary to ascertain preventive care for water meter after proper installation.

The ultrasonic and electromagnetic water meter's accuracy performance is assured for the life of the product. Turbine meter should be checked periodically for bearing wear since presence of air or particles in the water may damage the moving parts over time.

13.10.2 Preventive Maintenance

i. Proper handling, storage, and transportation of water meters
ii. To clean the dirt box or strainer wherever installed
iii. To replace the gaskets, if any
iv. To clean the chamber in which the meter is installed and keep free from flooding and seepage
v. To remove the meter for further internal repair/replacement if it does not show the correct reading pattern

13.10.2.1 Breakdown Maintenance

The only basic breakdowns observed during the periodical inspection are replacement of broken glass, lid, and fallen wiper, wherever provided. If a meter found not working, then it shall be removed immediately and sent to the meter service workshop. In meter workshops, the repair process typically involves several key steps.

i. Disassemble water meters including strainer, measuring unit, regulator, registering the device, etc.
ii. Clean all disassembled spare parts in detergent solution in warm water.
iii. Inspect the cleaned parts and replace worn parts and gaskets if any.

iv. Inspect the meter body spur threads and cover threads.
v. Inspect the sealing surface on the meter body and paint the meter body, if necessary.
vi. Inspect the vane wheel shaft pinion, bearing and pivot.
vii. Inspect the vane wheel chamber.
viii. Reassemble the water meter properly after reconditioning.
ix. Calibrate and test the repaired water meter for leakage and accuracy as per IS 6784: 1996 (Reaffirmed 2017).
x. Make an entry in the life register of that water meter for keeping history records.

The Table 13.5 gives remedies to common issues in a meter.

Table 13.5: Troubleshooting of water meter

S. No.	Trouble	Cause	Remedy
1.	Meter reads in reverse direction	Might have been installed in the reverse direction.	Check the arrow on the meter body and install the meter properly, if necessary.
2.	Meter not recording	Impeller to register link broken.	Remove the meter for servicing and repairs.
3.	Continuously moving pointer/digit rotates but no change in the indicator	Pointer and drum link missing. Drum defect.	Remove the meter for servicing and repairs. Remove the meter for servicing and repairs.
4.	Dial/glass foggy	Climatic condition.	Wait for climate change if it is the rainy season
5.	Meter suspected to be slow or fast	Inlet flow disturbance, missing internally defective, deteriorated magnets in case of magnetic meter.	Clean the external filter/dirt box, where provided, and the inbuilt strainer. Ensure full open condition of the upstream valve. If doubt persists, remove meter for testing, servicing and repair.
6.	Bush/gland leakage	Gland deformity.	Remove meter for testing and servicing.
7.	Regulator, head, body leakage	Regular washer damaged, loose screw.	Remove the meter and repair.
8.	Physical damage to meter including broken seal	Improper installation.	Remove meter for testing, servicing, and repair, physical protection arrangement be made.
9.	No water available past the water meter even though the inlet side is charged	Semi positive/positive displacement meter with jammed piston.	Meter is acting as a stop valve. Remove it for inspection, servicing, and repair.

In the case of smaller-sized water meters, it is advisable to check cost-benefit ratio before getting them repaired.

13.10.2.2 Prevention of Tampering of Water Meters

In order to prevent tampering, the following precautions should be taken:

i. The water meters shall be properly installed in the chamber or in C.I. covers equipped with locks and keys to avoid tampering.
ii. The water meters must be sealed properly.
iii. The water meter shall not allow reversible flow; it should register flow in forward directions only.
iv. The water meter dials should be easily readable without confusions.
v. The lids and glass of water meters must be made up of tough materials as per IS 779: 1994 and shall be replaced when necessary.
vi. Wiper or dial, as far as possible, is avoided.
vii. In the case of magnetically coupled meters, the proper material to shield magnets must be provided to avoid the tampering of such meter by outside magnets in the vicinity of the meter.
viii. Periodic inspection/checking at the site is essential to ensure the proper working of the meter.
ix. Special sealing arrangements may be necessary and provided for bulk meters whereby unauthorised removal of the meter from the connection can be detected.

Despite the above, to tackle the problems of tampering, suitable penalty provisions/clauses shall be there in the rules or the water supply agreement with the consumer. This will also discourage consumer tendencies of neglecting water meter safety.

13.10.3 Trend of Replacement of Water Meters

At present, there is no specific Indian certification process of validating the accuracy of water meters or flowmeter. In general, if a water meter goes out of order due to any physical damage or non-operation of the registration device and is beyond economical repair, it should be replaced with immediate effect. In the Indian context, the performance of water meter or flowmeter depends upon:

i. The quality of water meter produced by the manufacturer, and it differs from manufacturer to manufacturer;
ii. The design of pipeline and fittings in line with the meter;
iii. The workmanship and care when handling and installing the meter;
iv. The pattern of water passing through the meter;
v. The type of supply of water, whether it is continuous or intermittent;
vi. The meter maintenance, testing;
vii. The proper selection of meter; and
viii. Installation procedure as per {ISO 4064:2014 Part 5} to be followed.

The performance of a water meter is required to be watched continuously with suitable history sheets. Any abnormality noticed needs immediate action. Timely removal of faulty meter, especially mechanical ones, prevents cascading and cumulative damages.

Looking at the number of transactions involved, bulk meters shall be given priority in replacements. Based on the experience gained for a specification work, a well-planned programme for periodical meter testing, servicing, repairs, and replacement, wherever necessary, shall be designed.

13.11 Meter Reading Systems

13.11.1 Manual Meter Reading System:

Many of the mechanical meters do not have remote reading capability. The meter readers have to visit the consumer premises one by one, walk to the meter, and note down the registered readings manually. These readings are recorded manually in books or on cards and later fed manually to a customer accounting or billing system. In some cases, meter readers use handheld data entry terminals to record meter readings. Data from these devices are transferred electronically to a billing system. In other cases, the key entry has been replaced by mark-sense card readers or optical scanners.

The environment of meter reading usually is not favourable to the meter reader as most of the water meters are installed in the underground chamber; these chambers are filled, in many cases, with water, reptiles, or insects. Access to these meters is often obstructed when they are installed on consumers' premises. Sometimes, manual work is involved in opening the chamber covers. Some consumers connect their electrical earth terminal to water utility pipe which endangers the safety of the meter reader. If during the meter reading visit, the consumer premises are not accessible, the meter reader will have to visit it again, which increases the cost of meter reading. Alternatively, an "estimated" reading is used for billing, which can result in customer dissatisfaction due to high or low estimates.

13.11.2 Automatic Meter Reading (AMR) System

In AMR system, a meter reader carries a handheld device with a built-in or attached radio transceiver to collect meter readings from an AMR-capable water meter, generally called "smart meters". In this, the meter readings are automatically collected without manual intervention. This metering data is then transferred to a central computer either through cellular network or through USB cable for analysis and billing purpose. Readings from AMR can be obtained by a simple walk-by or drive-by method, where the meter reader either walks or drives down the street while automatically downloading the meter data. Figure 13.14 represents automatic meter reading (AMR).

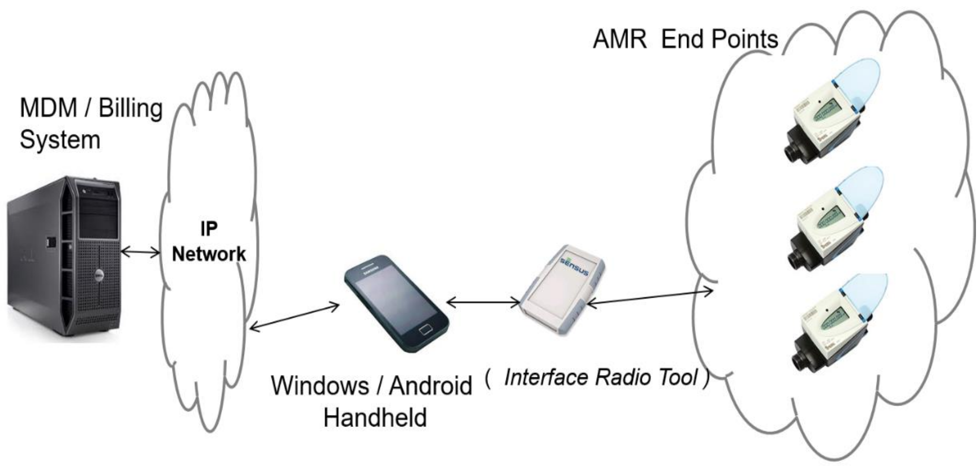


Figure 13.14: Representation of Automatic Meter Reading (AMR)

In AMR system, the meter readers read the meters either on monthly or bi-monthly basis as per the utility billing schedule. AMR system is much safer for meter readers since readers are not required to enter consumer properties, which in turn reduces the likelihood of injury. Due to the infrequent schedule of collecting data, any problems in the meter may go unnoticed for a significant period of time.

Benefits ascribed to the installation of an AMR system:

i. Increased revenue from previously unaccounted water;
ii. Accurate meter reading, no human error, and efficient billing;
iii. Reduced meter reading costs including both regular cycle reading and special reads;
iv. Improved safety of meter readers;
v. Increased customer service;
vi. Identifying and locating losses (customer and system);
vii. Theft detection;
viii. Improved cash flow;
ix. Conservation/Efficiency improvements;
x. Readings from data log containing register reads as frequently as every 15 minutes;
xi. Drastic reduction of estimated readings.

13.11.3 Advanced Metering Interface (AMI)

Many utilities have used the AMR system as a stepping-stone between standard metering and AMI systems. Some utilities made changes as technology became available, while other utilities made system decisions for financial reasons, and a few other utilities wanted to try out the technology as pilot before committing to an AMI system. However, if the goal is to eventually have a full AMI system, then it is smart to explore the cost-effectiveness of growing from AMR to AMI.

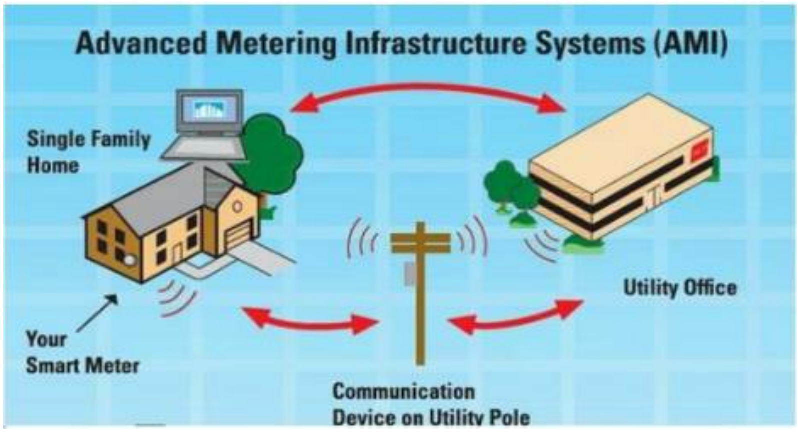


Figure 13.15: Advanced Metering Interface (AMI)

AMI comprises of smart meters and adds two-way communication between the meter and utility, and between the meter and consumer. This implies that in addition to providing readings, the meter can also receive (and often act on) instructions sent from the utility or consumer. Figure 13.15 and Figure 13.16 shows advanced metering interface (AMI).

AMI is more complex than AMR and requires a large physical communication network. AMI performs the function of data collection similar to an AMR system; however, instead of holding the collected data until a meter reader can collect it, AMI relays the data to the owner of the meter in real-time. Because AMI can relay data in real-time and has a physical network, it has additional features.

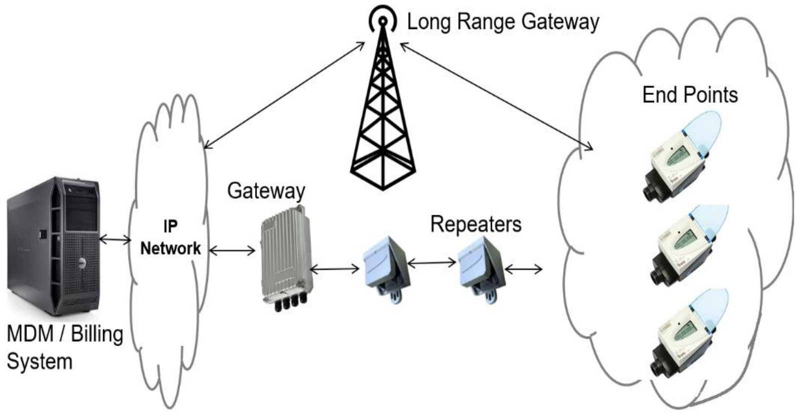


Figure 13.16: Representation of Advanced Metering Interface (AMI)

An AMI system can perform the functions of an AMR system as well as:

i. Daily, hourly, or 15-minute read increments without reading staff;
ii. Improved customer service;
iii. Customer web portals;
iv. Quick and accurate replies to inquiries;
v. Faster resolution of billing disputes;
vi. Select billing date;
vii. Real-time diagnostic operation and maintenance reports;
viii. Targeted data collection and report generation;
ix. Operational updates for the collector, repeaters, and endpoints;
x. Faster leak detection in water network with analytical module.

Data is transmitted in AMI systems in the following ways: one-way, two-way, and quasi two-way. The most appropriate option will largely depend on how much data is collected and the requirement of the utility.

13.11.4 Methods of AMI Data Transmission

13.11.4.1 Radio Technologies:

Because of its reliability and cost-effectiveness, radio frequency (RF) is the most common communication technology for AMI/AMR systems. Antennas or transmitters are attached to the meter or register, and data is transmitted from the meters and the data collectors by RF.

13.11.4.2 Non-Radio Technologies:

Non-radio technologies for data transmission include power lines, cable, cellular, satellite, telephone, GSM/GPRS, LoRaWAN , LAN, NBIOT, IIOT, optical fibre cable, etc.

13.11.4.3 Meter Data Management:

Meter Data Management (MDM) is a common platform where any smart meter can directly communicate and report meter reading data, alarms, tamper alerts, etc. to the user. The MDM is a cloud-based application capable of communicating with smart meters via different communication protocols. In the case of non-communicating meters, users can use android applications to collect data and send it via a mobile network to MDM for analysis, visualisation, and accuracy checks.

13.12 Model Compliance Sheet for Meter Tenders – AMR/AMI

The Model Compliance for AMR/AMI Meters is given as per the below Table 13.6

Table 13.6: Model Compliance Sheet for Meter Tenders – AMR/AMI

Sr. No.	Particulars	Compliance (YES/NO)	Documentary proof Page no.
1	Meter Measurement technology (Mechanical or Electromagnetic or Ultrasonic), IP68, facility for report communication interface	YES/NO	
2	Meter approvals and certification • OIML R 49 and or ISO 4064 • MID Module B + D for International /imported meters • Legal metrology Manufacturing plant – Quality Certificate • ISO 9001	YES/NO YES/NO YES/NO YES/NO	
3	Performance and Endurance Certificate from FCRI is submitted for each size of meter	YES/NO	
4	Operating range ratio – Q3/Q1 -R 160 minimum for mechanical and R 400 minimum for smart meters	YES/NO	
5	Meter Warranty from Manufacturer – 5 Years for AMR with mechanical meters or 10 years for Static Meters.	YES/NO	
6	Meter inbuilt battery (more than 5 years for mechanical meters and with more than 10 year battery life for static meters)	YES/NO	
7	Is Manufacturer Authorisation Form (MAF) from meter manufacturer submitted along with the bid	YES/NO	
8	Meter Manufacturer India Presence • India Local entity • Foreign entity • Local Service Support from OEM	• YES/NO • YES/NO • YES/NO	
9	Meter and Communication System (AMI) solution from same meter	YES/NO	

Sr. No.	Particulars	Compliance (YES/NO)	Documentary proof Page no.
	manufacturer or AMI system fixed network/LPWAN/NB-IoT		
10	AMI system of meter manufacture/ reading device/meter reading software/ gateways/ concentrator/ repeaters/ radio frequency / protocols/ battery life/ LPWAN/IoT is under warranty for 10 years O&M	YES/NO	
11	Is the Billing system offered capable of integration with AMR/AMI meters, water billing, supports collection, regularisation, grievance handling, Consumer information and integrate with meter and data management, GIS, Payment Gateway, MIS reports, customer portal, that will allow the Concessionaire/contractor to tailor the product to the specific working environment of the Municipality	YES/NO	
12	Is Smart Water Meter mechanical/electromagnetic/ultrasonic with certifications as required with facility for remote communication interface, battery-operated and warranty for 5 years for mechanical and 10 years for Electromagnetic/ Ultrasonic water meter proposed by the concessionaire / contractor. The mechanical/electromagnetic/ultrasonic water meter should maintain its accuracy +/-% over its lifetime and the measuring units should be m^3 for volume and m^3/h or l/h for flow rate	YES/NO	
13	Is the smart Water Meter mechanical/electromagnetic/ultrasonic Manufacturer proposed by the concessionaire/contractor has supplied in India or abroad at least 50,000 Static AMR/AMI Meters in one (1) lot of any sizes in last seven years	YES/NO	
14	Is the Smart Water Meter mechanical/electromagnetic/ultrasonic Manufacturer proposed by the concessionaire have manufacturing	YES/NO	

Sr. No.	Particulars	Compliance (YES/NO)	Documentary proof Page no.
	capacity to supply 1 lac meters in two years or as per needs of ULB.		
18	Is the concessionaire/contractor in agreement to the condition precedent scope in six months of GIS based consumer survey, commercial data validation, asset study, network study for gateways installation, ward wise consumer regularisation and new application execution and come out with projection of total water that can be supplied to potential water volume that can be sold	YES/NO	
19.	Is the concessionaire/contractor in agreement to set up a customer relationships management centre with test bench and participate with Municipality in the citizen engagement programme	YES/NO	
20.	Is the Concessionaire/contractor in agreement to the performance-based KPIs during the O&M period of 5 years for mechanical and 10 years for Electromagnetic/ Ultrasonic water meter	YES/NO	
21	For Mechanical/ electromagnetic/ ultrasonic meters weather proof mountable cabinet is provided for the electronics that should not cause obstruction to the RF/GSM/GPRS signal	YES/NO	
22	Meter proposed by concessionaire/contractor should provide three-point calibration with calibration certificate available for each unit. Agreeable for inspection by Municipality/ ULB before despatch	YES/NO	
23	Meter proposed by the concessionaire/contractor should be tamper proof and should have advanced diagnostics with indicators/alarm on display for tampering, reverse flow, and leakage	YES/NO	
24	All the supplied smart water meters, their peripherals and equipment's must	YES/NO	

Sr. No.	Particulars	Compliance (YES/NO)	Documentary proof Page no.
	have a written warranty from the manufacturers covering not less than 5 years for mechanical and 10 years for Electromagnetic/ Ultrasonic water meter from date of commissioning		

The characteristics of different types of meters with communication technologies are given in below Table 13.7.

Table 13.7: The characteristics of different types of meters with communication technologies

Characteristics	Non AMR-Mechanical Meters	AMR Meters-Mechanical	Advanced AMR-Mechanical	Ultrasonic AMR Water Meters	Electromagnetic Water Meters – AMR	NB-IOT Ultrasonic Water meters
Cost	X	3X	6.5X	10X	10X	10X
Product Lifetime	Upto 5 years	upto 5 years	7-8 Years	10-12 Years	10 -15 years	10-15 Years
Battery Life (Based on One Communication per Month)	NA	4-5 Years	7-8 Years	10-12 Years	10-12 Years	10-12 Years
Battery Life (Based on One Communication per Day)	NA	4-5 Years	10 Years	10-12 Years	12-15 Years	12-15 Years
Battery Life (Based on One Communication per hour)	NA	3 Years	5 Years	10-12 Years	12-15 Years	12-15 Years
Consumption Index Storage Frequency	NA	Daily and hourly Index available while fetching data through AMR	Daily and Hourly Index available	Daily Index and Hourly Index available	Daily and Hourly Index available	Daily and Hourly Index available
Consumption index recording capability	NA	Daily and hourly Index available	Daily and hourly Index available	Daily and Hourly Index available	Daily and Hourly Index available	Daily and hourly Index available

Interoperability (Capability to convert AMR to AMI and vice versa)	NA	Not possible	Yes	Yes	Yes	Yes
Accuracy (R=Q3/Q1) As per MID certification (Accuracy for all meter type will change in Intermittent Water Supply)	Upto R50	Upto R 80	Upto R160	Upto R800	Upto R800	Upto R800
Minimum flow rate	30 liters/hour	30 liters/hour	15 liters/hour	3 liters/hour	3 liters/hour	3 liters/hour
Nominal Flow rate	1600 liters/hour	2500 liters/hour	2500 liters/hour	2500 liters/hour	2500 liters/hour	2500 liters/hour
AMR Dataset capability to do online high resolution water balance and leak estimatio	NA	Hourly Water Balance Possible , Leak estimation not possible	Hourly Water Balance Possible and Leak Estimation Possible without affecting battery life	Hourly Water Balance Possible and Leak Estimation Possible without affecting battery life	Hourly Water Balance Possible and Leak Estimation Possible without affecting battery life	Hourly Water Balance Possible and Leak Estimation Possible without affecting battery life

13.13 Flowmeters

The flowmeter is the device used for the measurement of liquids in closed conduits. This device differs on the type of liquid conductive or non-conductive, and also have the other related aspects of the principle of operation. In water supply, mechanical, electromagnetic, or ultrasonic types of flowmeters are used. However, those are segregated depending on various points, i.e., working principle, conductivity of liquid and its quality, the basic and overall accuracy of the flowmeter, calibration possibility, reading are taken from field or online, etc. The supply and delivery manufacturer should have ISO quality standard (IS 9001:2015) certification and flowmeter testing confirming to ISO 17025: 2005 (Reaffirmed 2017). There are installation standards that need to be adopted for different flowmeters. Figure 13.17 shows electromagnetic flowmeter, Figure 13.18 shows ultrasonic insertion flowmeter, and Figure 13.19 shows ultrasonic clamp-on flowmeter.

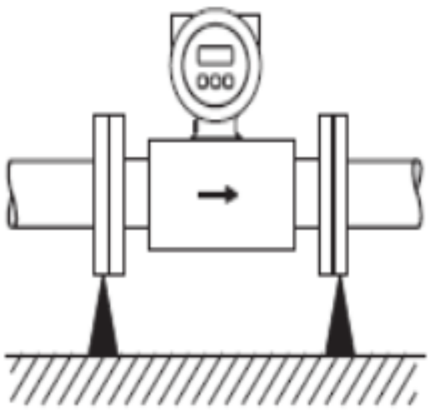


Figure 13.17: Electromagnetic flowmeter

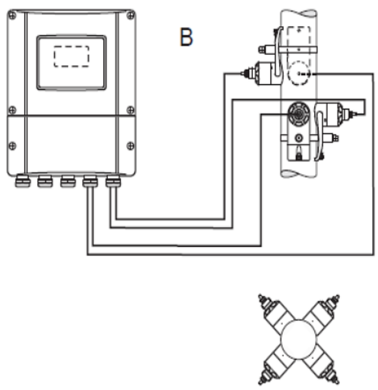


Figure 13.18: Ultrasonic Insertion flowmeter

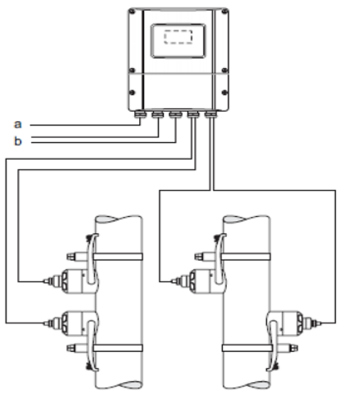


Figure 13.19: Ultrasonic Clamp-on flowmeter

13.13.1 Methods for Metering Flow

Various methods are available for the metering flow rate and total flow. Each method has its own specific characteristics, which are directed towards individual installation requirements. In the water industry, flow rate meter is termed as the flowmeter and the total flowmeter is termed as the water meter. A wide range of standard terms is used to describe the essential performance characteristics of instruments and sensors. Some of these terms are as follows.

13.13.1.1 Accuracy

It is defined as the difference between the reading of an instrument and the true value of the measured variable expressed as a percentage of either full scale or true value of the measured variable, i.e., either in terms of full scale or flow rate of the flowmeter. As far as possible, the accuracy should be selected in terms of percentage of flow rate as it remains constant within the rangeability irrespective of variation in flow rate.

13.13.1.2 Range

The difference between the maximum and minimum values of the physical output over which an instrument is designed to operate normally.

13.13.1.3 Rangeability/Turndown Ratio

Describes the relationship between the range and the minimum quantity that can be measured.

13.13.1.4 Linearity

The degree to which the calibration curve of a device matches a straight line.

13.13.1.5 Resolution

The error associated with the ability to resolve the output signal to the smallest measurable unit.

13.13.1.6 Repeatability

The quantity which characterises the ability of a measuring instrument to give identical indications or responses for repeated applications of the same value of the quantity measured under stated conditions of use.

13.13.2 Types of Flowmeters

In water works, normally, the following types of flowmeters are used. These are classified with their advantages and disadvantages as described in the Table 13.8.

Table 13.8: Types of Flowmeters

S. No.	Types of Flowmeter	Advantages	Disadvantages
A	**Differential Pressure/Head Flowmeter**		
1.	Orifice Flowmeter	i. It can be used for all fluids except for some exceptions ii. No moving parts iii. Flow rate, indication, integration is easily obtained iv. It can be fitted in any configuration of the pipeline v. Suitable for any pipe diameter vi. The signal can be transmitted to long distance vii. Good accuracy viii. Suitable for extreme temperature and pressure ix. Calculation possibilities for unusual situations	i. Rangeability 4:1 ii. Energy cost in terms of head loss iii. Ideal conditions are required for good accuracy iv. Suitable for a particular range of Reynolds number v. Accuracy in terms of span vi. Minimum slope for tapping piping has to be maintained, i.e., 1:10 vii. Very long conditioning section required viii. Intensive maintenance required ix. Edge sharpness of the orifice must be assured x. It requires isolation of pipeline during installation

S. No.	Types of Flowmeter	Advantages	Disadvantages
2.	Venturi Meter	Advantages are similar to orifice flowmeter, and less pressure loss and hence less energy cost.	Advantages are similar to orifice flowmeter mentioned at Sr. No. i, iii, iv, v, vi, and x in addition to the high capital cost
3.	Pitot Tube	i. As mentioned under orifice flowmeter except at Sr. No. vii. ii. It does not require isolation of pipeline for installation and comparatively capital cost of the flowmeter is less. iii. Head loss is also less.	As mentioned under orifice flowmeter at Sr. No. i, iii, v, vi, vii with addition of inferiority in accuracy as it being point velocity measurement
4.	Annubar (Average Pitot Tube)	Similar as mentioned under pitot tube in addition to higher accuracy	Similar as mentioned under pitot tube except for inferiority in accuracy, i.e., accuracy improves due to averaging of multi-ported pressures.
B	**Linear Flowmeter**		
1(a)	Turbine Wheel Flowmeter (Full Bore or Inline)	i. Excellent accuracy, linearity, and repeatability ii. Usable at extreme temperature and pressure	i. Suitable for only for low viscosity ii. Moving parts and hence, wear iii. Sensitive to contamination iv. Flow profile sensitive and needs conditioning section v. Affected by overloading, the danger of overspeeding vi. Sensitive to vibration vii. Isolation of pipeline is required for installation.
1(b)	Turbine Wheel Flowmeter (Insertion Type)	i. Isolation of pipeline is not required ii. Low cost	i. Inferior accuracy because of point velocity measurement ii. Suspended impurities can clog it iii. In addition to the above, the disadvantages mentioned under the turbine wheel flowmeter (full bore) are also applicable

S. No.	Types of Flowmeter	Advantages	Disadvantages
2	**Variable Area Flowmeter (Rotameter)**		
	Variable Area Flowmeter (Rotameter)	i. Inexpensive ii. No power supply required for local indication iii. No conditioning section iv. Easy maintenance	i. It requires vertical installation ii. Affected by the density and temperature of the fluid iii. Affected by vibration and pulsation
3	**Vortex Flowmeter**		
3(a)	Full Bore or Inline Type	i. No moving part ii. Robust construction iii. Unaffected by temperature, pressure, and density changes	i. Conditioning of long approached section ii. Span limitation due to viscosity iii. Shedding rate is nonlinear between 2000 and 10000 Reynolds's number iv. Available up to 400 mm size due to constraints of sensitivity v. Isolation of pipeline is required for installation
3(b)	Insertion Vortex Flowmeter	i. Isolation of pipeline for installation is not required ii. Less costly than that of full bore iii. In addition to the above, the advantages mentioned under full bore vortex flowmeter are also applicable.	i. Inferior accuracy due to point velocity measurement ii. In addition to the above, the disadvantages mentioned under full bore vortex meter are applicable except at Sr. No. V.
4	**Magnetic Flowmeter**		
4(a)	Full Bore (Inline) Flowmeter	i. Unobstructed flow passage ii. No moving parts iii. No additional pressure drop iv. Unaffected by changes in temperature, density, viscosity, electrical conductivity v. Flow range setting can be optimised vi. Suitable for water containing suspended solids vii. Short conditioning section is required as it is insensitive to flow profile	i. Air or gas inclusion causes the error ii. Minimum required conductivity of fluid 0.5 ms/cm. iii. Isolation of pipeline is required for installation iv. Vacuum creation may detach inner liner

S. No.	Types of Flowmeter	Advantages	Disadvantages
		viii. Measures flow both the directions ix. Unaffected by contamination and deposit x. Minimum maintenance xi. Good linearity xii. The smaller diameter flowmeter can be used on a bigger diameter pipe with the help of reducers having angle not more than 16 degrees	
4(b)	Insertion Magnetic Flowmeter	i. Less costly than that of full bore ii. No isolation of pipeline for installation iii. Advantages mentioned under Sr. Nos. ii, iv, v, vi, viii, ix, x, xi of full bore (inline) magnetic flowmeter is applicable	i. Inferior accuracy due to point velocity measurement ii. Long conditioning section is required iii. Sensitive to vibration iv. Periodic cleaning of the electrode is required v.
5	**Ultrasonic Flowmeter**		
5(a)	Doppler Type Ultrasonic Flowmeter	i. Unobstructed flow passage ii. No moving parts iii. No pressure drop iv. Measures flow in both directions v. Installations of individual elements in existing pipe lines possible vi. Minimum maintenance vii. Economical for large diameter pipe viii. Suitable for turbid water	i. Not suitable for clear water ii. Accuracy is inferior iii. It requires a long conditioning section
5(b)	Transit Time (Time of Flight) Ultrasonic Flowmeter	i. Advantages mentioned under Sr. nos. i, ii, iii, iv, v, vi, vii of Doppler type are applicable ii. Accuracy is improved in multipath iii. Accuracy is superior in insertion (wetted type) than that of clamp type.	i. It requires long conditioning section ii. Not suitable for turbid water or carrying air/gas bubbles.

13.13.2.1 Ultrasonic Flowmeters

Ultrasonic flowmeters utilise the properties and behaviour of sound waves passing through moving water. The ultrasonic flowmeters are of two types depending on different working mechanisms, viz., transit time meters and Doppler meters.

13.13.2.2 Transit Time Ultrasonic Flowmeters

Transit time ultrasonic flowmeters are based on the phenomenon that sound waves slow down when moving through the water against the flow, and speed up when they move with the flow. A transit time ultrasonic flowmeter has two sound transducers mounted at opposite sides of the pipe at an angle to the flow. Each of these sound transducers will in turn transmit an ultrasound signal to the other transducer. The differences in the transit times of the signals determine the flow velocity and flow rate.

The accuracy of the transit time ultrasonic flowmeters depends on the ability of the meter to accurately measure the time taken by the ultrasound signal to travel between the sound transducers. Larger pipes have longer path lengths, and thus, the speed of the signal and the flow rate can be measured with higher accuracy. Transit time meters work better in clean fluids and thus are ideal for drinking water pipes. They measure the average velocity of fluid but are sensitive to the velocity profile in a pipe. In some cases, multi-beam devices are used to improve meter accuracy.

Permanently installed transit time meters are often called wet transducer meters since their sound transducers are in direct contact with the fluid. These meters are very reliable. Ultrasonic flowmeters typically have relative errors between 0.25% and 1%. They can be used on pipes ranging from 10 mm to greater than 2 m in diameter, although they are not often used on small diameter water pipes. The ideal flow velocity range for good accuracy is 0.5 to 10 m/s.

The inline type meter, with IP 68 protection class, is constructed as a vacuum chamber of moulded composite material or suitable metal body. Thus, the electronics are fully protected against penetration of water. Water consumption is measured electronically, as a volume, using the ultrasound signal. Through two ultrasonic transducers, an audio signal is sent with and against the flow direction. The ultrasonic signal travelling with the flow will be the first to reach the opposite transducer, while the signal running against the flow will be received a little later. The time difference between the two signals can be converted into flow velocity. The measuring principle is called 'bidirectional ultrasound technique based on the transit time method', which is a proven, long-term, stable, and accurate measuring principle.

Clamp-on transit time meters use sound transducers that are clamped externally onto the walls of a pipe to provide portable non-intrusive flow measurement. Practically, they can be used on any pipe material including metals, plastics, fibre, cement and lined or coated pipes. A disadvantage of ultrasonic flowmeter is that the ultrasonic pulses must traverse pipe walls and coatings, and therefore the thicknesses and acoustic properties of these elements must be known. Deposits on the inside pipe surface can affect signal strength and performance. Additionally, air in the liquid, turbulence, deposits on the sensors, and water hammer (pressure transients) affects its performance.

Modern clamp-on meters incorporate microprocessors that allow mounting positions and calibration factors to be calculated for each application and can provide accuracies of 0.5% to 2%. The advantages of transit time flowmeters include high accuracy and reliability, which makes them cost-effective for use in large pipes. The clamp-on version of the meter is easy to install without the need to shut down the pipe. However, transit time flowmeters are sensitive to distortions in the velocity profile of a pipe, require an electricity supply, and are not suitable for dirty waters.

13.13.2.3 Doppler Ultrasonic Flowmeters

Doppler ultrasonic water meters function based on the Doppler Effect, which is the measure of the change in the frequency of a sound wave when it is reflected back from a moving object. Doppler ultrasonic flowmeters create a soundwave in a moving fluid which, upon contact with dirt particles or air bubbles, reflects back towards the origin of the signal. The reflected ultrasound waves are detected by a receiver, and the change in the wave frequency is measured. This shift can then be related to the velocity and thus flow rate of the water. Doppler ultrasonic flowmeter is shown in Figure 13.20.

They can only be used for water that contains particles or air bubbles, and thus they are more suitable for dirty water applications such as raw water. A drawback of Doppler meter is that fluid particles in the water sometimes move slower than the water itself, or are concentrated in parts of the pipe with lower velocities (e.g., close to the sides or bottom of the pipe), which can result in a measurement error of 10% or more. They are also sensitive to disturbances in the velocity profile and require an electrical supply. While they are not suitable as billing meters, they can be cost-effective as flow monitors if measurement accuracy is not critical.

Figure 13.20: Doppler Ultrasonic Flowmeter

13.13.2.4 Sensor Based Flowmeter

Remote flowmeter, for use on gravity fed and pressurised piped water systems, is designed to monitor functionality and performances of water supply networks. The pipe flowmeter utilises the ultrasonic flowmeter, microprocessor, and a body trace chip to monitor water usage on piped systems. The sensor has a plastic enclosure waterproof, anti-explosion, heat and cold resistant, and utilises food-compliant plastic.

After installation on the water pipe, when the water tap is opened, the flow of water passing through the sensor results in the rotation of the internal turbine. The energy of the fluid passing through it moves a rotor that have magnets on his blades; the volume of the water is registered by monitoring the speed of the rotation of the magnets passing by a metal point. This information is then transformed in digital data by the microprocessor, and the data sent by the sim card embedded in the sensor. The sensor transmits hourly flow data that can be uploaded on cloud platforms. Dashboards have to be designed according to the user needs. This is an advance technology and may be applied in water supply systems.

Meters shall also have an actual flow rate and totalised value for effective water management purposes. The accuracy shall be ±0.5% of reading for flowmeters.

The supplier shall have full ISO 9000 series accreditation and fully traceable calibration methods. The suppliers shall also have a testing facility in India so that methodology and procedures can be verified. Each flowmeter shall be wet calibrated with a two-point calibration to verify performance in accordance with the specification and submit the report for the same. The testing facility shall be duly accredited in accordance with ISO/IEC 17025:2005 (Reaffirmed 2017) Standards. Bidders must upload/attach the certificate of ISO/IEC 17025 from flowmeter manufacturer as a mandatory requirement of this enquiry/tender which is duly accredited according to ISO/IEC 17025:2005 (Reaffirmed 2017) facilities in India.

The sensors for flowmeters shall be as per DVGW/ISO standard lengths (ISO 13359:1998) so that interchangeability can be carried out for the applicable flowmeter sizes. The sensor shall also have built-in grounding and empty pipe detection electrodes of SS 316 for detecting partial flow conditions and efficient operation purposes. The liner material shall be polyurethane (PU) or hard rubber suitable for media/application/service. The appropriate certificate for drinking water approval shall be a part of it and the same shall be uploaded or attached while bidding as a mandatory requirement of this tender. The sensor and transmitter shall be capable of working in a tropical environment. The flowmeter body shall be available in flanged or with custom connectors as specified in the datasheets.

The sensors shall be rated IP 68. The transmitter shall be rated IP 67 in line with local operating conditions. Installations shall be made with cables and/or conduits that guarantee the integrity of the system under all operational conditions. The transmitter/converter shall be the wall-mounted type with a two-line display for the indication of an actual flow rate and totalised value. A glass window within the protection enclosure with optical switches shall be provided for local reading purposes. The non-corrosive, polycarbonate housing material of the enclosure shall be sufficient to guarantee five years of operational life. Magnetic flowmeters should be supplied with built-in software features to analyse and continuously monitor the health of the sensor, display errors in text format. The transmitter should be capable of performing the verification programme on demand or on request without taking the meter off the line or without any additional external hardware/accessories.

The transmitter shall be capable of being fully programmable. It shall have a set-up menu so that all relevant parameters may be user-set from the self-prompting driven menu. The transmitter shall have three totaliser units, one scalable pulse output, and one current, i.e., 4–20 mA HART output. The current output shall be galvanically isolated. It shall be fitted with a switched-mode power supply capability of 0–250V or 24 DC and 45–65Hz to cope with power transients without damage. The totaliser value shall be protected by EEPROM during a power outage and utilises an overflow counter.

The transmitter shall be having the facility of indicating electrical conductivity measurement. It shall be possible to separate the sensor and transmitters up to 300 meters without the need for signal boosters or amplifiers. The pulsed DC type flow sensors shall normally be installed remotely from the transmitters and are to be subject to harsh environmental conditions. At some locations, underground chambers shall be used and, in such cases, the operation under fully submerged conditions may occur. Thus, in either case, a full IP68 design is necessary. The sensor shall, therefore, be made from SS 304 materials with flanges of up to PN10 rating from carbon steel as per EN 1092-1, suitably treated for the application. The sensor coil housing shall be powder-coated cast aluminium with NEMA 4X rating (IP 68) or painted steel. The paint shall be of durable anti-corrosion grade. The tube liner shall be suitable for media/application/service.

The manufacturer shall have a full system of local offices in India and full-service capability in the metro cities throughout the country. Full contact details for key personnel, both national and local shall be furnished on request. The supplier shall provide evidence of at least five years of involvement in the manufacturing of meters worldwide. The water flowmeter manufacturer/supplier shall provide full data on each meter required, including optimising and sizing programmes calculation sheet. The proposed flowmeter model number by the manufacturer shall be available on their official website with a complete technical catalogue or operating manual for flowmeter (sensor /transmitter). The official latest meter sizing programme shall be available on the official website of the flowmeter supplier. The proposed model code shall be available and acceptable globally.

13.13.3 Installation and Maintenance of Flowmeters

Every user expects a problem-free installation of the meter and thereafter only accurate reading. Regular monitoring is desirable in order to avoid failures.

It is essential to install the flowmeter co-axially to the pipeline without protruding any packing or gasket into the water flow stream. In the case of ultrasonic flowmeter, the probes are welded on the pipeline which requires care to see that no projection is protruding in the pipeline.

The meter is installed in the pipeline using flanged or threaded connections giving due consideration for conditioning sections. It should be seen that stress-free installation is carried out in pipeline. It is essential to install the flowmeter co-axially to the pipeline without protruding any packing or gasket into the water flow stream. In the case of ultrasonic meter, the probes are welded on the pipeline which requires care to see that no projection is protruding in the pipeline. In this case, onsite calibration is essential. Whenever converters are used with primary elements, it should be observed that the connection between them should be protected against lightning strikes and any other interference signal.

The installation on the existing water supply requires shutting down the water supply. This necessitates shortest installation time. The installations are strictly carried out as per manufacturers' recommendations.

In the case of differential pressure type flowmeter, the impulse piping requires special care in respect of slope and protection. Similarly, long disturbance free straight sections should be provided for uniformity. Installation should be vibration free as moving parts in the flowmeter, wherever present, will get worn out in addition to the effect on overall accuracy of the flowmeter. Installation in 'U' shape is essential for intermittent water supply.

Flowmeters should be provided with battery backup in order to retain integrator reading during failure of electric supply.

13.13.3.1 Repairs, Maintenance, and Troubleshooting of Flowmeters

Modern development in the flowmeter measurement is that in most of the equipment a self-monitoring facility is provided with which the maintenance staff monitors the health of the equipment. A number of instruments are enunciating the error conditions.

As far as orifice, Pitot tube, Venturi, and Annubar flowmeters are concerned, they require regular purging of impulse piping. Similarly, the transducers require periodical checking of zero and range setting. For the orifice, it is essential to check sharpness of the edge as in the case of its deterioration or damage the flowmeter reading may vary up to 20%.

Ultrasonic flowmeters and magnetic flowmeters are self-monitoring provide information regarding deviation in accuracy or failure of probe or electrode. Whenever cleaning of probes or electrodes is required, those should be cleaned as per manufacturers' recommendation.

Turbine meter should be periodically checked for bearing wear since presence of air in the liquid may damage the bearing because of over speeding.

Where deposits are to be expected in any flowmeter, the same should be regularly inspected and cleaned as per the experience gained during the course of time. As these deposits affect the accuracy of the measurement, vortex meter, magnetic flowmeter, and ultrasonic flowmeter may show erroneous reading in the presence of deposits. Average accuracies of various flowmeters are given in Table 13.9. In an intermittent water supply, the corrosion rate of the pipe increases due to chlorine and air. The formation of incrustation and subsequent descaling affect flowmeter working especially differential pressure type and turbine meters.

Table 13.9: Average Accuracies of Various Flowmeters

Sr. No.	Type of flowmeter	Accuracy %
1.	Square edge orifice	±1S
2.	Venturi	±1S
3.	Pitot	±2S
4.	Annubar	±1S
5.	Turbine	±0.5R
6.	Rotameter	±2S
7.	Vortex	±1R
8.	Magnetic	±0.5R
9.	Doppler	±2S
10.	Transit time	±1R

Legends: S: in terms of full scale; R: in terms of flow rate.

Table 13.10 gives broad areas of Application of Flowmeter for Liquid

Table 13.10: Broad Areas of Application of Flowmeter for Liquid

Sr. No.	Type of flowmeter	A	B	C	D
1.	Orifice	0	(+)	0	0
2.	Venturi	0		0	0
3.	Variable Area	0	0		
4.	Annubar	0		0	0
5.	Turbine	0		0	(*)
6.	Insertion turbine	0		0	0
7.	Vortex	0			
8.	Insertion Vortex	0		0	0
9.	Electro Magnetic	0	0	0	0
10.	Insertion Electro Magnetic	0		0	0
11.	Doppler	0		(+)	(+)
12.	Transit time	0	(+)	0	0

Legends: 0: Suitable, generally applicable; C: Large liquid flows (>1.7 × 10^4 L /min.); (+) is worth considering, sometimes applicable; (*) is worth considering, limited availability or tends to be expensive; D: Large water pipes (> 500 mm dia); A blank indicates unsuitable; liquids (temp.>200^0C) not applicable; A: General liquid application (< 50 CP); B: Low liquid flows (<2 L /min)

A flowmeter suspected to be malfunctioning is also tested for its accuracy of the measurement. The testing is done as per {IS 6784: 1996 (Reaffirmed 2017)}/ {ISO 4064-2014 Part III}. A faulty flowmeter, if found to be repairable, is repaired and tested and calibrated for its accuracy before installation. To establish a meter factor, the indicated volume of fluid that passes through a meter is compared to the true volume measured in a container of known size, or a master meter. Temperature and pressure correction are then applied.

The flowmeter manufacturer/supplier shall provide full data on each meter required, including optimising and sizing programmes calculation sheet. The proposed flowmeter model number by the manufacturer shall be available on their official website with a complete technical catalogue/operating manual for flowmeter (sensor/transmitter). The official latest meter sizing programme shall be available on the official website of the flowmeter supplier. The proposed model code shall be available and acceptable globally.

The orifice, Pitot tube, Venturi, and Annubar flowmeters require regular purging of impulse piping. Similarly, the transducers require periodical checking of zero and range setting.

Suggest changing to "In the case of deterioration or damage to the orifice edge, it is essential to check its sharpness, as it can lead to flowmeter reading variations of up to 20%.

They are generally self-monitored and give information regarding deviation in accuracy or failure of probe or electrode. Whenever cleaning of probes or electrodes is required, those should be cleaned as per manufacturers' recommendations.

Where deposits are to be expected in any flowmeter, the same should be regularly inspected and cleaned as per the experience gained during the course of time. As these deposits affect the accuracy of the measurement, vortex meter, magnetic flowmeter, ultrasonic flowmeter, may show erroneous reading in the presence of deposits. In an intermittent water supply, the corrosion rate of the pipe increases due to chlorine and air. The formation of incrustation and subsequent descaling effect flowmeter working especially differential pressure type, turbine meters.

13.13.3.2 Flowmeter Calibration

There are two philosophies of flowmeter calibration. One is that it is better to have a fixed calibration system with all the associated technical back up and with the flowmeters being brought to the calibration system; the other favours calibrating in situ leaving the flowmeters in their installed condition and using a portable calibrator. The former will generally provide the more accurate calibration, but the latter has the advantage of accounting for site-specific effects, such as proximity to hydraulic disturbances. It is necessary to decide carefully which option to adopt.

There is often no choice but to carry out in situ calibration where:

- flow cannot be shut off;
- site-specific conditions have to be accounted for;
- the meter is so large that removal, transport, and testing costs would be prohibitive.

The major constraint with the in situ calibration technique is that the high accuracy laboratory calibration cannot be matched in the field and accuracies of ±2% to ±5% is all that can be achieved, and such field tests are called confidence checks rather than absolute calibrations. Such checks are often the precursor to the removal of flowmeter for laboratory calibration or replacement.

For field tests, the following methods can be used:

i. Clamp-on devices
ii. Thermodynamic method
iii. Velocity-area methods (insertion meters)
iv. Tracer methods

v. Flow simulators

Normally, the manufacturers of the flowmeters provide laboratory calibration of the flowmeters in their works. Some of the Government agencies also provide laboratory calibration, vis., Fluid Control Research Institute (FCRI), Palakkad, Central Water and Power Research Station (CWPRS), Pune and Institute for Design of Electrical Measuring Instruments (IDEMI), Mumbai.

Table 13.11 gives the performance factors of flowmeters and Table13.12 gives installation constraints of flowmeters. Table 13.13 gives fluid property constraints for flowmeters. Table 13.14 gives economic factors of flowmeters. The installation methods for various types of flowmeters along with its maintenance and service is discussed in Table 13.15.

Table 13.11: Performance Factors of Flowmeters

Sr. No.	Type of the flowmeter	Linearity %	Repeatability (%)	Rangeabilit y	Pressure drop at maximum flow	Flow parameter measured
1.	Orifice	0.25% FS to 1% FS	±0.2% FS	3 or 4:1	3-4	R
2.	Venturi	0.25% FS to 1% FS	±0.2% FS	3 or 4:1	2	R
3.	Variable area	±1% FS to ±5% FS	±0.5% FS to ±1% FS	1% FS	10:1	3R
4.	Annubar	0.5% R to 1% R	±0.05% R to ±0.2% R	4 to 10:1	1/2	Vm
5.	Turbine	±0.15% R to ±1% R	±0.02% R to ±0.5% R	5 to 10:1	3	R
6.	Insertion Turbine	±0.25% R to ±5% R	±0.1% R to ±2% R	10 to 40:1	1-2	Vp
7.	Vortex	±1% R	±0.1% R to ±1% R	4 to 40:1	3	R
8.	Insertion Vortex	±2% R	±0.1% R	15 to 30:1	1	Vp
9.	Electro Magnetic	±0.2% R to ±1% R	±0.1% R to ±0.2% FS	10 to 100:1	1	R
10.	Insertion Elec. Mag.	±2.5% R to ±4% R	±0.1% R	10:1	1	Vp
11.	Doppler	No data	±0.2% FS	5 to 25:1	1	Vm, R
12.	Transit time	±0.2% R to ±1% R	±0.2% R to ±1% FS	10 to 300:1	1	R

Legends: R: Flowrate, Vp: Point velocity, NS: Not specified; T: Volume flow; % R: Percentage flowrate; 1: Low; Vm: Mean velocity; % FS: Percentage full scale; 5: High

Table 13.12: Installation Constraints for Flowmeters

Sr. No.	Type	Orientation	Direction	Quoted range of upstream lengths	Quoted range of minimum downstream	Pipe Diameter (mm)
1.	Orifice	H, VU,VD,I	U,B	5D/80D	2D/8D	6 to 2600
2.	Venturi	H,VU,VD,I	U	0.5D/29D	4D	>6
3.	Variable area	VU	U	0D	0D	2 to 150
4.	Annubar	H, VU,VD,I	U,B	2D/25D	2D/4D	>25
5.	Turbine	H, VU,VD,I	U,B	5D/20D	3D/10D	5 to 600

Sr. No.	Type	Orientation	Direction	Quoted range of upstream lengths	Quoted range of minimum downstream	Pipe Diameter (mm)
6.	Insertion turbine	H, VU,VD,I	U,B	10D/80D	5D/10D	>75
7.	Vortex	H, VU,VD,I	U	1D/40D	5D	12 to 400
8.	Insertion Vortex	H, VU,VD,I	U	20D	5D	>200
9.	Electromagnetic	H, VU,VD,I	U,B	0D/10D	0D/5D	2 to 3000
10.	Insertion magnetic	H, VU,VD,I	U,B	25D	5D	>100
11.	Doppler	H, VU,VD,I	U,B	10D	5D	>25
12.	Transit time	H, VU,VD,I	U,B	0D/50D	2D/5D	>4

Legends: H: Horizontal flow; U: Unidirectional flow; VU: Upward vertical flow; B: Bidirectional flow; VD: Downward vertical flow; D: Inner diameter of pipe; I: Inclined flow.

Table 13.13: Fluid Property Constraints for Flowmeters

Sr. No.	Type	Maximum pressure (bar)	Temperature Range (˚C)	Minimum Reynold's number	More than one phase (Gas or liquid)
1.	Orifice	400	<650	3×10^4	P
2.	Venturi	400	<650	10^5	P
3.	Variable area	700	-80 to + 400	No data	N
4.	Annubar	400	<540	10^4	N
5.	Turbine	3500	-260 to +530	10^4	N
6.	Insertion Turbine	70 to 250	-50 to +430	10^4	N
7.	Vortex	260	-200 to +430	2×10^4	P
8.	Insertion Vortex	70	- 30 to +150	5×10^3	N
9.	Electromagnetic	300	-60 to +220	No limit	S/P
10.	Elect. Insertion	20	+5 to +25	No data	N
11.	Doppler	Pipe pressure	-20 to +80	5×10^3	S
12.	Transit time	200	-200 to +250	5×10^3	N/P

Legends: S: Suitable; P: Possible; N: Not suitable

Table 13.14: Economic Factors of Flowmeters

Type	Installation cost	Calibration cost	Operation cost	Maintenance cost	Spares cost
Orifice	2-4	1	3	2	1
Venturi	4	1-4	2	3	3
Variable area	1-3	2	2	1	1
Annubar	2	3	2	2	2
Turbine	3	4	3	4	4
Insertion Turbine	2	3	2	2	3
Vortex	3	3	3	3	3
Insertion Vortex	2	3	2	3	3
Electromagnetic	3	3	1	3	3
Insertion Ele. Mag.	2	3	2	3	2
Doppler	1-3	1	1	3	2
Transit time (time of flight)	1-3	3	1	3	2

Legends: 1: Low; 5: High

Table 13.15: Installation and Maintenance of Flowmeters

Type	Installation	Pipeline ahead of meter	Maintenance during operation	Self-Monitoring	Service
Turbine meter	Flanged connections electrical installation	Conditioning section	Maintenance free, monitor, possible foreign lubrication	Not possible	—
Vortex meter	Flanged connections or water installation, electrical	Conditioning section installation	Maintenance free	Error monitoring	Electronic monitor functions and test values
Differential pressure Meters	Primary in flanges, impulse piping, convertor power supply	Long conditioning sections	Regular monitoring	Not possible	Direct measurement at primary
Variable area	Flanged or threaded	No restrictions	Maintenance free	Constant appearance	—

Type	Installation	Pipeline ahead of meter	Maintenance during operation	Self-Monitoring	Service
meter	connections				
Electromagnetic flowmeter	Flanged connections, electrical connections	No conditioning section	Maintenance free	Monitoring with error announcements	Electronic control functions and test simulator
Ultrasonic meter	Flanged connections or welding nipples, electrical installation.	Long conditioning section	Maintenance free	Signals for signal loss	-

13.13.4 Problems Encountered in Flowmeter Performance

There are many problems which are encountered during the life of the flowmeter. Some of the common performance-related issues/problems that are encountered during its operation due to some causes along with remedial action are discussed in Table 13.16.

Table 13.16: Common Problems Encountered in Flowmeter Performance

S. No.	Problems	Causes	Flowmeter	Remedial Action
1.	Erratic reading	Operated below lower range having limited rangeability of flowmeter	Differential pressure type	Replace flowmeter
		Operated below lower range having limited rangeability of flowmeter	Linear flowmeter	Change range setting
		Less static pressure	D.P. type	Remove air trap
		Clogged impulse piping	D.P. type	Clear the choke up
		Air trapped in impulse piping	D.P. type	Remove air trap
		Frequent air trap in impulse piping	D.P. type	Change impulse piping slope to minimum 1:10, If still the problem persists change the flowmeter
		Damaged impulse piping	D.P. type	Rectify impulse piping
2.	Unsteady reading: (oscillating)	β ratio of more than 0.65	D.P. type	Redesign orifice
		Pulsating flow	D.P. and Linear type	Condition the flow
3.	Inaccurate reading	Pipeline internally incrusted	D.P. and Linear type	Clean the internal surface of pipeline
		Scaling is formed at tapping points	D.P. type	Clean the tapping points
		Orifice edge gets blunt	D.P. type	Replace orifice plate
		Flowmeter downstream is opened within the range of 50 times dia pipe length	D.P. type	Extend the downstream pipeline beyond 50 times dia length
		Unsymmetrical formation of vena contract due to the large diameter of the throat in relation to static pressure	D.P. (orifice type)	Redesign the orifice
		Mismatch between flowmeter and pipeline	D.P. and Linear type	Remove the mismatch
		Absence of sufficient conditioned approach pipeline	D.P. and Linear type	Provide sufficient conditional approach pipeline
		Foreign particles such as pieces of concrete, bricks, debris. etc. are gathered at upstream of the orifice	D.P. (Orifice)	Remove them
		Flanged coupling used with flowmeter leaking	D.P. and Linear type	Rectify the leakage
		The pipeline may not be	D.P. and	Replace the pipe length

S. No.	Problems	Causes	Flowmeter	Remedial Action
		cylindrical within the range of 0.3% of the diameter of the pipe	Linear type	of 2 times dia immediate upstream of the flowmeter
		Pipeline partially filled	D.P. and Linear type	Install valve downstream of the flowmeter for throttling

13.13.4.1 Calibration of Pressure Measuring Instruments

Pressure instrument calibration is the process of adjusting the instruments output signal to match a known range of pressure. All instruments tend to drift from their last setting. This is because springs stretch, electronic components undergo slight changes on the atomic level, and other working parts sag, bend, or lose their elasticity.

The calibration procedure includes zero, span, and linearity adjustments. The pressure is varied with the help of a pneumatic calibrator so as to give desired pressures to the instrument. The settings are carried out on the instrument for zero and span adjustment on the basis of applied pressures. For carrying out linearity, setting various pressures between zero and maximum range of the instruments are applied and adjusted the output of the measuring instrument with the help of controls provided in the instrument.

In the case of pressure gauges, the calibration is carried out by means of dead weight tester. In the absence of a pneumatic calibrator, the air can be supplied to the instrument with proper pressure regulator, and pressure is measured with the help of a manometer so as to calibrate the instrument. The calibration should be checked every 3, 6, or 12 months depending upon the use and accuracy expected, as per the manufacturer's recommendation and latest ISO standards. Maintenance of pressure instruments is essential for their proper working and accurate reading. It also improves the life and reliability of the instruments.

13.13.4.2 Preventive Maintenance

The manufacturer of the instrument gives the instructions in the manual supplied along with the instruments. These instructions explain how to maintain the instrument. Generally, these consist of the following categories:

1. **Visual Inspection:**
   Any damage to piping or wiring of the instrument observed should be immediately rectified. It avoids the entry of foreign bodies into the system and further damage to the instrument.
2. **Venting or Blow down**
   Liquid lines are generally clogged subsequently if those are not vented periodically. Similarly, air or gas in the liquid columns gives wrong readings. In order to avoid such incidents, it is essential to blow down the instrument piping periodically on the basis of experience gained in the field.
3. **Cleaning and Lubrication**
   Instruments with mechanical linkages undergo wear and misalignment. Dirt may clog the linkages, causing the mechanism to become less flexible. If not attended these kinds of faults, the instrument may breakdown subsequently. This clogging can be removed by cleaning, and the working of the instrument can be improved by lubrication as per manufacturer's recommendations. Dust can be removed from the panels as well as from the instruments with the help of an air blower. If auto test facility is provided on the instrument by the manufacturer, the same can be used to check the performance of the instrument daily. If any kind of fault occurs, in such instrument, the same is identified and displayed by the instrument itself. A typical troubleshooting

chart for pressure and level measuring instrument (electronic transmitter type) is given in Table 13.17 and typical specification for online measurement of pressure is given in Table 13.18.

Table 13.17: A Typical Troubleshooting Chart for Pressure and Level Measuring Instrument (Electronic Transmitter Type)

Fault	Possible Causes	Corrective Action
Low output or zero output or High output or Erratic output	Power Supply	Check the output of power supply
		Check for short and multiple grounds
		Check polarity of connections
		Check loop impedance
	Pressure tapping	Check the pressure connection
		Check for leakage or blockage
		Check for entrapped air or gas in the line
	Transmitter	Check for shorts in sensor leads
		Check connector to transmitter
		Check for amplifier assembly by replacing it with the spare one
	Sensing element	Check the sensing element for its working by gently tapping it
Tapping by hand gently the mechanism sensor does not respond	Mechanical	Check mechanical linkage
		Check for dirt finding
		Excessive wear, misalignment
		For dirt clean and lubricate as per manufactures recommendations
		Realign mechanical parts if necessary
		For wear replace the worn-out components
	Electrical	Replace electrical/electronic subassemblies and perform calibration

Table 13.18: Typical Specification for Online Measurement of Pressure

Parameters	Descriptions
Specifications	Pressure Range 0 to 10/20/50/100 Kg/cm^2 g
Process Temperature Range	-20 to + 125 °C
Output Signal	4 to 20 mA with superimposed digital communication protocol HART, 2-wire
Signal Range	4 to 20 mA HART
Signal on Alarm	As per NAMUR NE 43 • 4 to 20 mA HART Options: Max. alarm: can be set from 21 to 23 mA (factory setting: 22 mA) a) Hold measured value: last measured value is held b) Min. alarm: 3.6 mA
Resolution	Current output: 1 microAmp Display: can be set (setting at the factory: presentation of the maximum accuracy of the transmitter)
Response Time	<250 ms
Damping	Via local display, handheld terminal or PC with the operating programme, continuous from 0 to 999 s

Parameters	Descriptions
	Additionally, for HART: via DIP switch on the electronic insert, switch position "on" = value set in the 2 Sec As default
Supply Voltage	11.5 to 45 V DC
Residual Ripple	No influence on 4 to 20 mA signal up to ±5% residual ripple within the permitted voltage range [according to HART hardware specification HCF_SPEC-54 (DIN IEC 60381-1)]
Influence of Power Supply	< 0.0006% of URL/1 V
Reference Accuracy	±0.075% of the set Span
Climate Class	Class 4K4H (air temperature: –20 to 55 °C/–4 to +131 °F, relative humidity: 4 to 100%) satisfied as per DIN EN 60721-3-4 – Seal Capsuled electronics.
Housing	Die Cast Alu. Housing
Diaphragm Material	Ceramic Dry measuring cell, Capacitive measuring cell
Display Operation	3 Key Push button for configuration without HART COMMUNICATOR. A 4-line liquid crystal display (LCD) is used for display and operation
Long-Term Stability	±0.25% URL/year for 1 year, 5 year
Turn Down	100:1

13.13.4.3 Radar Level Transmitters

They provide a non-contact type of level measurement in case of liquids in a metal tank. They make use of EM, i.e., electromagnetic waves usually in the microwave X-band range which is approximately 10 GHz. Hence, they can be also known as microwave level measurement devices. However, there are some differences between radar and microwave types. They are:

i. Power levels in the case of radar systems are about 0.01 mW/cm^2, whereas in the case of microwave systems, these levels range from 0.1 to 5 mW/cm^2.
ii. Microwaves can work at higher energy levels; hence, they are competent enough to endure extra coating as compared to radar level detectors.

A radar level detector includes:

i. A transmitter with an inbuilt solid-state oscillator
ii. A radar antenna
iii. A receiver along with a signal processor and an operator interface

The operation of all radar level detectors involves sending microwave beams emitted by a sensor to the surface of the liquid in a tank. The electromagnetic waves, after hitting the fluid's surface, return back to the sensor which is mounted at the top of the tank or vessel. The time taken by the signal to return, i.e., Time of Flight (TOF) is then determined to measure the level of fluid in the tank. The specifications of Radar Level Transmitter is given Table 13.19.

Table 13.19: Radar Level Transmitter

Radar – Microwave type level transmitter	Service: Raw Water (Non-Contact Type)
Transmitter	
Type	Microwave level measurement
Principle	Pulse Time of Flight

Radar – Microwave type level transmitter	Service: Raw Water (Non-Contact Type)
Output	4–20 mA HART Current
Housing	Die Cast Aluminium
Electromagnetic compatibility	Interference Immunity to EN 61326, Annex A (Industrial) and NAMUR
Ingress protection	IP65/IP 66/IP 67
Accuracy	±3 mm
Area classification	Non-Hazardous
Display	4-line LCD Display. Menu guided operation. Display of Envelope Curve.
Configuration	Using the keypad on display
Sensor	
Range	Liquids 0 to 5 m and 0 to 10 m depending on Tank size
Temperature range	-40 °C ... +80 °C
Max pressure	3 bar abs
Materials	Sensor: PVDF Seal: EPDM
Antenna seal	FKM Viton
Process connection	Threaded or universal flange dependent on model selection
Degree of protection	IP65

13.13.5 Telemetry and SCADA Systems

13.13.5.1 Manual Monitoring

Normally, the managers of O&M of water utilities monitored levels in service reservoirs, pressures, and flows in a distribution system and on the operation of pumps such as hours of pumping and failure of pumps and monitored water quality by measuring residual chlorine. The manager usually used the telephone line or wireless like VSAT or GPRS/GSM unit to gather the data, used his discretion gained with experience, and took decisions to ensure that the system is operating with the required efficiency. Manual collection of data and analysis is an outdated practice and may not be helpful in large undertakings if water utilities have to aim at enhanced customer service by improving water quality and service level with reduced costs. This is possible if the management acquires operational data at a very high cost.

13.13.5.2 Telemetry

The inspection, monitoring, and control of O&M of a water utility can be automated partially through telemetry. Telemetry enables regular monitoring of the above data on a real-time basis and the data is provided to anyone in the organisation who can review the data and make a decision. In a telemetry system, probes/sensors will be used which will sense and generate signals for the level, pressure, flow, and water quality like pH, turbidity, residual chlorines in a given unit, and transmit the signals by radio/by telephone, VSAT, GSM/GPRS. Normally, radio link is used and telephone line with the modem is used as spare communication. Microwave satellite or fibre-optic transmission systems are also used for data transmission. The water pumping stations may communicate via a cable buried with the pipe. However, there may be locations where the main power may not be available and, hence, solar panels with a battery charger are used to power the remote terminal unit (RTU) and the radio, VSAT, GSM/GPRS. In urban areas, RTUs can communicate on cell phones and or packed radio networks. For remote locations satellite technology is also available.

i. Data for Collection by Telemetry

The data includes levels in service reservoirs, pressures and flows in a distribution system, flows/quantity and water quality like pH, turbidity, residual chlorines of delivery into a service reservoir and data on the operation of pumps such as voltage, amperes, energy consumed, operating times and downtimes of pumps and chlorine residuals. In a telemetry system, up-to-the-minute real-time information is gathered from a remote terminal unit located at the water treatment plant, reservoir, flowmeter, pumping station, etc., and transmitted to a central control station where the information is updated, displayed and stored manually or automatically.

ii. Processing Data from Telemetry

The meter readings from reservoirs are useful information for managing the distribution system and help in preventing overflow from reservoirs. However, the effectiveness of telemetry in pumping operations is dependent on the reliability of instrumentation for measuring flows, pressures, KWh meters, etc. Standard practice is to calculate pump efficiency and water audit calculations on a monthly basis. Telemetry can also be used to supervise a water hammer protection system wherein the pump failures are linked to initiate measures to prevent the occurrence of water hammer.

13.14 SCADA Systems

Instead of manual review of data collected by telemetry and initiating action manually, if telemetry is extended to include actions based on the data for remote control of pumps and other equipment, then such a system is known as supervisory control and data acquisition (SCADA). SCADA is a computer-aided system that collects, stores, and analyses the data on all aspects of O&M. It gives a better understanding of what is happening in terms of water quantity or water quality which is sourced and supplied. SCADA can do any activity like on/off any equipment or start /stop.

The operating personnel can retrieve the data and control their operations and sometimes the system itself is programmed to control the operations on the basis of the acquired data. SCADA enhances the efficiency of the O&M personnel who are better informed about the system and hence are in full control of the operations. Whether in a telemetry system or a SCADA system, up-to-the-minute real-time information is gathered from the remote terminal unit located at the water treatment plant, reservoir, flowmeter, pumping station, etc., and transmitted to a central control station where the information is updated, displayed, and stored manually or automatically. In a SCADA system, the information is linked to a supervisory system for local display, alarm annunciation, etc., which may be linked to remote control of pumping operations or operation of valves, etc.

13.14.1 Data Collected in SCADA/Smart Metering System

SCADA systems will have probes/sensors which will sense and generate signals for the level, pressure, and flow in a given unit, and transmit the signals for storage and analysis on the computer. The signals are transmitted by radio, by telephone, microwave satellite, or fibre-optic transmission systems. SCADA systems can include the network diagrams of the distribution system of which detailed sketches of a particular area can be viewed by the operator if necessary to observe the current operating data such as flow, pressure, level, or residual chlorine.

SCADA systems in water distribution are programmed for the collection and processing of the following information.

i. To monitor levels in service reservoirs, pressures and flows in a distribution system;
ii. To monitor and store data on levels in SRs, or flows/quantity of delivered into SR or pressures of the distribution system and generate alarms for threshold values of levels, flows and pressures to initiate operation of valves and pumps;
iii. To monitor and store data on the operation of pumps such as voltage, amperes, energy

consumed, operating times and down times of pumps;

iv. To measure and record chlorine residuals and generate alarms at thresh hold values of residual chlorine in the distribution systems.

13.14.2 Analysis of Data from SCADA/Smart Metering

SCADA systems can be designed to analyse the data and provide daily, weekly, monthly, and/or annual reports or schedules. It also helps in monitoring the inventories on spare parts and plan requirement of spares. Responses for different scenarios such as seasonal changes or any emergencies can be programmed into SCADA. The information stored in the SCADA can be easily retrieved and analysed. Typical information that could be generated in the system includes consumption patterns linked to the weather conditions, plots on pressures against flows, electrical energy consumption linked to consumer demands, record on system leaks, record on pump failures, areas with fewer chlorine residuals, etc. A typical SCADA system architecture and dashboard in shown in figure 13.21 & 13.22.

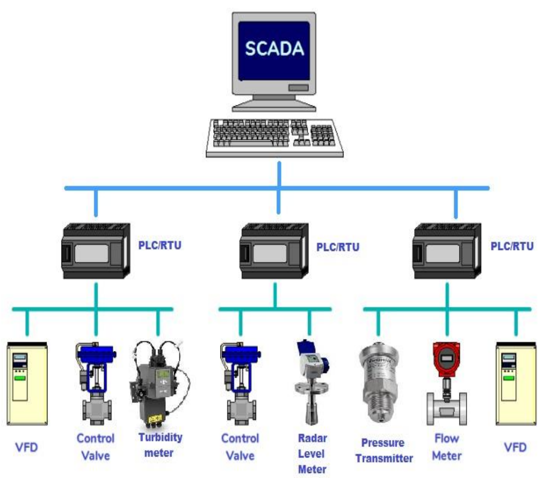


Figure 13.21: SCADA System Architecture

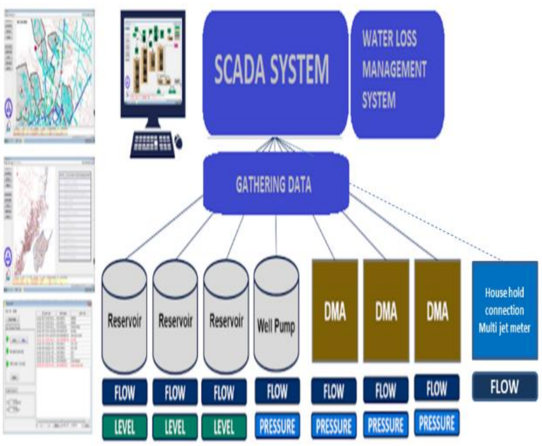


Figure 13.202: SCADA System Dashboard

13.14.3 Limitations of SCADA/Smart Metering/Communication

Before installing a SCADA, the utility staff should visit facilities with SCADA and discuss with the utility managers and then decide the scope of SCADA to be provided in their utility. The objective of SCADA should be to make the job of operator easier, more efficient, and safer to make their facilities performance more reliable and cost-effective. There is no doubt that SCADA enables better capacity utilisation and help in improved service levels at low operating cost. SCADA designing calls for careful planning and requires a phased implementation, particularly dependent on appropriate training of utility staff and their willingness to adopt the new technology.

The availability of power supply is very essential to the efficient functioning of the system. Wherever possible, the RTU for flowmeter or pressure sensor and water quality like pH, turbidity, residual chlorine is provided power from electricity mains via a battery that acts as a buffer in case of mains failure. There may be metering locations for flow and pressure sensors without any source of power close by. In such cases, solar power may be one alternative. Initially installations at such locations may operate well but they are always subject to poor after-sales service by vendors, vandalism, and theft.

Ultimate improvement in the water supply distribution system cannot be achieved through the advanced application of technology like SCADA. The utility staff should have reached a reasonable level of managerial capabilities even with conventional methods of monitoring and control by adopting a holistic approach when the SCADA may further enhance their capabilities; SCADA by itself is not the answer for poor or inefficient management.

13.15 Conclusion

The present field measurement market, which caters to the water quantity and quality, is open and there are many new technologies available based on the requirement of the purchaser. If the right instrumentation for measurement and control like a water meter, flowmeter, etc., are selected and installed properly, the maintenance cost, manpower cost, etc., will get reduced drastically and accuracy also will be maintained.

It is also pertinent to mention that there is no clear data available in the country regarding the quantity of water drawn or abstracted from the source or treated in a treatment plant or consumed by consumers. As the precious water resources are getting scarce and depleted day by day, it is essential to adopt technologies for ensuring proper measurement, maintaining quality, and conservation. By adapting the necessary field instrumentation devices, water balance, NRW, etc., can be known and necessary steps may be taken for better management of water. The correct and accurate measurement of inputs and outputs need to be given due weightage and importance in all water utilities for effective and productive utilisation of precious potable water resources by way of water audit and energy audit.

There is a need for judicious use and conservation of limited availability of water, hence it is important that a demand management programme is introduced in each ULB by expanding the consumer water metering programme. It is essential for every ULB/Water Board to have a metering policy similar to the example set by Chennai Metro Water and Sewerage Board. This water meter policy provides the objective, roles and responsibilities of all stakeholders and the technical standards for the water metering programme. The consumers are classified as domestic, partly commercial, commercial, industrial, institutional, and municipal bulk supply, based on the nature of water usage activity of the consumer. The policy should cover types of consumers for metering, selection of meter, its specification and installation guidelines, and in addition, should cover the tariff, billing and collection, and grievance redressal procedures related to metering. Further, the responsibilities of the ULB/Water Board and consumers regarding metering can be described. The consumer meters shall be owned by the Board and fixed rental charges to be levied on the consumers along with the water charges. The rental charges shall be used for the specific purpose of routine and periodical maintenance of the meters and regular meter replacement programme by the ULB/Water Board. The day-to-day safety and upkeep of the meters shall be the responsibility of the consumer. If any wilful tampering or damage of the meters occurs other than natural wear and tear, the ULB/Water Board will undertake replacement of such defective meter and the cost of such exceptional repair or replacement of meter shall be recovered from the consumer with advance notice. Having a metering policy will also enable the ULB/Water Board to have back-to-back long-term warranty with various meter manufacturers.

To make the ULB/Water Board sustainable, a tariff for water supply has to be fixed for each category of consumers for metered and unmetered premises. Preferably, the tariff has to be designed in such a way that a long-term tariff for water supply is fixed for different category of consumers based on volumetric tariff with lowest tariff for domestic, full cost recovery for commercial, industries and should include institutional, municipal bulk supply, and mobile water supply.

In summary, the key objectives of the tariff and metering policy would be to

a) promote water conservation by encouraging efficient water use;
b) reduce non-revenue water (NRW) and increase cost recovery;
c) ensure fairness and equity to all Consumers in charging for water services;

d) achieve 100% of metering to all households including all high rise and commercial connections;
e) set out roles and responsibilities of both ULB/Water Board and consumer in relation to the installation of metered connections, maintenance of the water meters.

One of the key advantages of implementing a metering system to improve efficiencies of a ULB/Water Board is that it lends itself for a Public-Private Partnership (PPP) under the Hybrid Annuity Model (HAM) suggested under AMRUT 2.0 guideline where viability gap funding is available. The HAM can be structured to a performance-based contract with partial funding by private concessionaire, wherein a long-term performance-based O&M can be structured comprising of fixed and variable payments linked to performance such as efficiency of redressal of customer complaints, meter reading, billing and distribution efficiency, response time to new water supply connections, and functioning of water meters.

General specifications to be followed for water meters to sustain 24×7 water supply system:

- In case of static meters, the water meters should be Ultrasonic or Electromagnetic with shelf life of atleast 10 years, the battery life of these should not be less than 10 years.
- In case of mechanical multijet meters or mechanical multijet AMR, the minimum shelf life shall be of 5 years and the battery life should not be less than 5 years.
- The suppliers shall be made responsible for O&M for 10 years in respect of static meters and 5 years in respect of mechanical meters including for repair and replacement of meters.
- Water meters should have FCRI and OIML/MID certification and approval for legal metrology. Such certificate should be issued in the name of the manufacturer supplying the meters.
- Water meter should be approved/certified by FCRI for ISO 4064-1 and ISO 4064-3 of 2014.
- Water meter should be IP68. In case of electro-magnetic and Ultrasonic meters it should be R 400 and above. In case of mechanical meters with smart communication AMR/AMI it can be R160 due to limitations.
- Smart meters/ static meters should enable NB-IoT/LoRaWAN/Walkthrough/Drive through for data reading.
- Water meter should provide one reading per day and should have internal memory to store meter reading for 90 days.
- Performance tests related to influence factors and disturbances is mandatory for electromagnetic / ultrasonic static meters and all type of meter manufacturers need to provide conformity assessment with respect to quality assurance of production process with technical documentation and also type design approval with endurance certificate as conducted by FCRI for module approval program (MAP) is mandatory for long life product.

If the ULB is proposing for smart water meters in water supply systems, the following standards (Table 13.20) shall be followed:

Table 13.20 Applicable Standards for Water Meters and Flowmeters

S.No.	Standard/Reference	Title/Description
General		
1	BS: 7405:1991 confirmed year: 2017	Selection and application of flowmeters for the measurement of fluid flow in closed conduits
2	BS: 5792:1980 Replaced by: BS EN ISO 20456:2019	Specification for Electro Magnetic flowmeters
3	BS EN ISO: 6817:1997 Replaced by: BS EN ISO	Measurement of conductive liquid flow in closed conduits – Method using Electromagnetic flowmeters

S.No.	Standard/Reference	Title/Description
	20456:2019	
4	ISO Recommendation: R-541: 1967(E) Replaced by: ISO 5167-1:2003	Measurement of fluid flow by means of orifice plates and nozzles
5	ISO 9104:1991/ Revised by: ISO 20456:2017 BS 7526: 1991	Measurement of fluid flow in closed conduits — Method of evaluating the performance of electromagnetic flowmeters for liquids
6	BS: 6199: 1991/ ISO 9368:1990 Confirmed Year:1998	Measurement of liquid flow in closed conduits by using weighing and volumetric methods
7	IS: 4477 Part-2: 1975 Reaffirmed Year: 2016	Methods of measurement of fluid flow by means of Venturi meters: Part-2 Compressible Liquids
8	IS 2951 Part I: 1965 Reaffirmed Year: 2017	Recommendations for estimation of flow of liquids in closed conduits Part I: Head loss in straight pipes due to frictional resistance
9	IS 14615 Part I: 1999: 2018	Measurement of fluid flow by means of pressure differential devices — Part I: Orifice plates, nozzles and venturi tubes inserted in circular cross-section conduits running full
10	IS 9115: 1979 Reaffirmed Year: 2017	Method for estimation of incompressible fluid flow in closed conduits by Bend meters
11	IS 779: 1994 Reaffirmed Year: 2015	Water meters (Domestic type) – Specification (Sixth revision)
12	IS 2373: 1981 Reaffirmed Year: 2017	Specifications for water meters (Bulk type) (Third revision)
13	IS 6784: 1996 Reaffirmed Year: 2017	Methods for Performance Testing of Water Meters
14	BS 5728	Measurement of flow of cold potable water in closed conduits Part I (1979): Specification for single meters Part II (1980): Specification for installation requirements for single meters Part III (1997): Methods for determining principal characteristics of single meters
15	ISO: 4064:2014 Confirmed Year: 2019	Water meters for cold potable water and hot water Part-1: Metrological and technical requirements Part-2: Test methods and equipment Part-5: Installation requirements
16	AWWA Manual 6	Water Meters – Selection, Installation, Testing and Maintenance
17	IS 17482:2020	Drinking Water Supply Management System — Requirements for Piped Drinking Water Supply Service
Closed Pipe Flow Measurements		

S.No.	Standard/Reference	Title/Description
1	ISO 1088: 2007 Confirmed Year: 2020	Velocity-area methods using current-meters — Collection and processing of data for determination of uncertainties in flow measurement
2	ISO 3354: 2008 Confirmed Year: 2017	Velocity-area method using current-meters in full conduits and under regular flow conditions
3	ISO 4006: 1991 Confirmed Year: 2019	Measurement of fluid flow in closed conduits — Vocabulary and symbols
4	ISO 4064-1: 2005 Replaced by: ISO 4064-1:2014	Measurement of water flow in fully charged closed conduits — Meters for cold potable water and hot water — Part 1: Specifications
5	ISO 4064-2: 2014 Confirmed Year: 2019	Water meters for cold potable water and hot water — Part 2: Test methods
6	ISO 4064-5: 2014 Confirmed Year: 2019	Water meters for cold potable water and hot water — Part 5: Installation requirements
7	ISO 4185: 1980 Confirmed Year: 2019	Measurement of liquid flow in closed conduits — Weighing method
8	ISO 5167-1: 2003 Confirmed Year: 2014 Status: Current	Flow Measurement via Differential Pressure Methods: General
9	ISO 5167-2: 2003 Confirmed Year: 2014	Flow Measurement via Differential Pressure Methods: Orifices
10	ISO 5167-3: 2003 Confirmed Year: 2014	Flow Measurement via Differential Pressure Methods: Nozzles
11	ISO 5167-4: 2003 Confirmed Year: 2014	Flow Measurement via Differential Pressure Methods: Venturis
12	ISO 5168: 2005 Reaffirmed 2015	Measurement of fluid flow — Procedures for the evaluation of uncertainties
13	ISO 6416: 2004 Revised by ISO 6416:2017	Measurement of discharge by ultrasonic (acoustic) method
14	ISO/TR 9464: 2008 Confirmed Year: 2019	Guidelines for the use of ISO 5167
15	ISO 6817: 1992 Replaced by: ISO 20456:2017	Closed Pipe Flow Measurements: Electromagnetic Flowmeters
16	ISO 8316: 1987 Confirmed Year: 2019	Flow Measurement by Volumetric Tank Collection Method
17	ISO 9104: 1991 Replaced by: ISO 20456:2017	Methods of evaluating the performance of electromagnetic flowmeters for liquids
18	ISO/TR 9824: 2007	Measurement of Free Surface Flow in Closed Conduits
19	ISO 11631: 1998 Confirmed Year: 2014	Methods for Specifying Flowmeter Performance
20	ISO/NP 12242: 2012	Measurement of fluid flow in closed conduits — Ultrasonic transit time meters for liquid
21	ISO 13359: 1998 Replaced by:	Flanged Electromagnetic Flowmeters: Overall length

S.No.	Standard/Reference	Title/Description
	ISO 20456:2017	
22	ISO/TS 25377: 2007 Confirmed Year: 2013	Hydrometric Uncertainty Guidance (HUG)

CHAPTER 14: AUTOMATION OF WATER SUPPLY SYSTEMS

14.1 Introduction

Water supply is a basic necessity for people; the gap between demand and supply is growing day by day. This imposes the need to practise optimum utilisation of water through the use of information technology at source, treatment, transmission, and distribution till it reaches the consumer's end. This implies continuous supervision of the water supply process in order to allow any problem that could appear to be isolated, analysed, and resolved, and at the same time, to maintain normal functioning parameters to the best possible level. Proper solutions imply effective point/stage wise monitoring with automation architectures including a supervision and control system for the real time sensing of various parameters, programmable logic controllers-based control system with basic functions (communication, adjusting, measuring, etc.) libraries, communication systems, standard interfaces or dedicated ones connected with sensors, electrical drive elements, measuring devices, etc.

The informatics systems present the possibility of monitoring, analysing and processing the data, leading to an optimum functioning of the stage as well as the entire water supply system. This chapter presents a philosophy of automation system for the monitoring, control, and predictive analysis of the process parameters in the water supply system for efficient energy usage, reduction of water losses and optimum administration of the water supply system.

14.2 Purpose and Objective

Application of new technology and 'smart (real-time process parameter measurement)' devices is well-established in all type of works for sustainable services, including water supply networks. Remote and online monitoring is one of the easiest ways to control flow and pressure among other water-related process parameters in water supply networks. In order to reduce losses in the network, online monitoring and controlling is more than a necessity. Water loss reduction is one of the biggest challenges today in the water supply system.

The automatic and electronic devices will help monitor and control leakages in the water supply systems. The application of these new technologies in water supply system eliminates not only water wastage but also provides continuous water flow at predetermined pressure range. The methodology involves automation with minimum human intervention leading to effective and efficient network distribution, cost effectiveness and maintaining safe drinking water quality parameters.

The sensor-based monitoring is not only to monitor key performance indicators viz., flow, pressure, leaks, residual chlorine, etc., but also to ensure early detection of faults/deviations from set routines, enable quick response, minimise service delivery outage, minimise water loss, optimise efficiency, and monitor the quantity and quality on sustainable basis. The additional advantage of this data would be to analyse the demand pattern of the user groups over time and use this information for demand management at appropriate level, minimise non-revenue water, ensure proper management and effective operation, and maintenance of water supply systems in the ULBs.

A combination of sensors, automation systems, and a cloud-based Internet of Things (IoT) system will be the most appropriate to provide a comprehensive and robust smart online water management system.

14.3 Instruments and Control Systems

Instrumentation and control refer to the analysis, measurement, and control of process variables using process control instruments and software tools through an automatic control system. Some of the more common process variables measured in a water distribution system includes flow, pressure, level, chemical dosage, water quality, energy, and temperature. Instrumentation should be selected, installed, commissioned, and maintained properly ensuring the data that it can give is invaluable to the water management authority/body.

The control system may be manual or automatic or a combination thereof. Regardless, the system should be designed to promote energy efficiency, conserve water, and reduce waste while meeting the treated water quality standards and demands under all anticipated conditions.

14.3.1 Methods of Control

The key characteristic of control is to interfere, to influence, or to modify the process. This control function or the interference to the process is introduced by an organisation of parts (including operators in manual control) that, when connected together, is called the control system. Depending on whether a human body (the operator) is physically involved in the control system, they are divided into manual control and automatic control (SCADA). Due to its efficiency, accuracy, and reliability, automatic control is widely used.

i. Manual

Manual Control system is an open loop control system. This type of system needs an external effort to adjust and correct the errors. Manual control system is less reliable. Manual controls are applicable when judgment and discretion are required. Additional risks arise with the use of manual controls as they can be more easily overridden, susceptible to human error, and are inherently less consistent than automated controls.

ii. SCADA

Supervisory control and data acquisition or SCADA is a type of process control system architecture that uses computers, networked data communications, and graphical human-machine interfaces (HMIs) to enable a high-level process supervisory management and control.

SCADA is a collection of hardware and software components. Plant floor devices such as pumps, valves, and transmitters transfer real-time data to processors such as remote terminal units (RTUs) or programmable logic controllers (PLCs). That data is then disseminated to various devices within the network such as HMI terminals, servers, and computers. Images of these processes are presented to operators for various types of interaction.

Generally, SCADA is done by relying on simple indicators such as lamps, analogue meters, or alarms. Along with the development of computer technology, the computer becomes an important component used in designing SCADA. However, an IoT system is more suitable for web-based monitoring and control for several reasons. These reasons are elaborated in the next sections.

In SCADA system, HMI is used as a display of connecting SCADA system between man and machine. HMI displays data to the operator and provides the input medium used to control the process by the operator. Without using HMI, the process of supervision and control will be more difficult and require more time.

The hardware components consist of pumps and motors, valves, switches, and transmitters. These components will communicate with a processor which, in turn, will allow the components to be viewed, graphically, on the SCADA system.

14.4 Internet of Things (IoT) System

Huge advancements in computing, storage, networking, and sensor technologies over the last decade or so has given rise to IoT.

At high level, IoT is a network of interrelated intelligent devices, computers, machines, and objects that are capable of transferring data without requiring human-to-human or human-to-computer interaction. The application of IoT in water distribution refers more specifically to interconnected control systems, sensors, instruments, industrial assets, computers, and people. IoT enables intelligent operations using advanced data analytics, resulting in transformational business outcomes. The scale and span of IoT can be massive, easily resulting in deployments reaching into the thousands, if not tens of thousands of individual endpoints. When properly utilised and optimised, it can be one of the largest enablers for smart systems.

14.4.1 IoT Architecture

As depicted in Figure 14.1, there are three tiers in an IoT system – Edge, Platform and Enterprise Application.

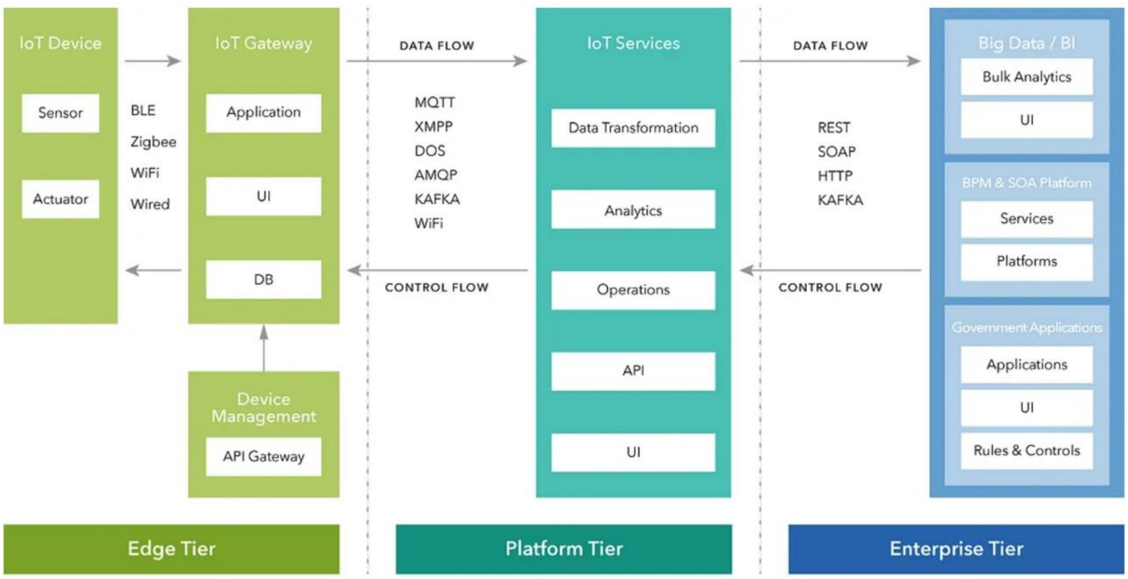


Figure 14.1: Industrial Internet Reference Architecture
Source: https://medium.com/@wiprodigital

14.4.1.1 Edge tier

Consists of IoT devices and the IoT gateway. The architectural characteristics of this tier, including its breadth of distribution and location, depend on the specific use cases of the enterprise. It is common for IoT devices to communicate using a relatively short range and specialised proximity network, due to power and processing limitations. The IoT gateway contains a data store for IoT device data, one or more services to analyse data streaming from the IoT devices or from the data store, and control applications. The IoT gateway provides endpoints for device connectivity, facilitating bidirectional communication with the enterprise systems. It also implements edge intelligence with different levels of processing capabilities.

14.4.1.2 Platform tier

This typically resides in cloud or some sort of internet/intranet and uses web server or similar infrastructure. It provides foundational services to Edge tier and Enterprise tier. Some services provided by platform are data processing and storage, stream analytics engine, alarm and alerts module, AI/ML engine, among others.

14.4.1.3 Enterprise/Application tier

This tier resides on top of the platform and shares platform services and data storage. It has the application for a specific requirement, e.g. CMS. It provides user interface and application specific analytics, dashboards, reports. It runs real-time analytics on the data and gives actionable alerts to users. Industrial control systems like SCADA and DCS are suitable for time-critical (microseconds latency) primary control of a process. In water distribution domain, WTP is an example of such a process. For other components like pumping station, ESR, FCV, time-critical control is not required. Control logic of such components may be based on data from other parts of the network in addition to the component being controlled. Thus, an IoT system where data from all parts of network is processed is suitable for non-time-critical control.

In short, IoT working on top of automation systems and integrated with technologies like AI/ML is an ideal digital platform for distributed water management system. To ensure every drop of water gets meaningfully utilised, India must embark on the journey to digitalise water distribution. The beginning of the transition to a centrally monitored water supply distribution and management system is the most important. It involves:

a. the monitoring and control requirements of the existing system;
b. understanding how available instruments/technology along with control systems can be used to achieve a real-time monitoring and control;
c. understanding the upgradations that needed in the existing water supply system; and
d. working on a cost-effective final solution for the utility.

14.5 Automation at Various Components of Water Supply System

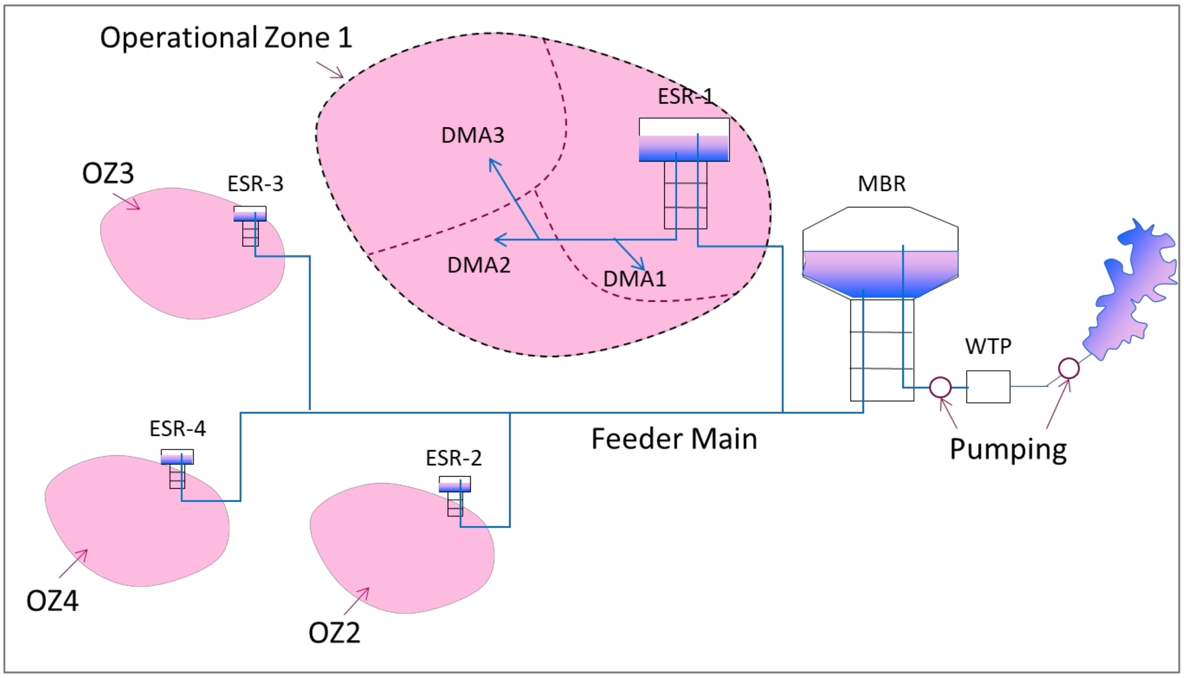


Figure 14.2: Typical water supply scheme

A typical water supply scheme is shown in Figure 14.2. The following components of a water supply system can be considered for the purpose of automation, real-time monitoring, and centralised control.

1) Water pumping systems
2) Water treatment plants (WTPs)
3) ESR/MBR/GSR
4) Key points in Distribution System

SCADA is suitable for the control of a plant (e.g., WTP, ESR, etc.) but not for distribution systems. IoT platform with a water distribution application is more suitable compared to SCADA. In a distribution system each device, each node, and each facility needs to be stitched together, and these will be very large numbers.

14.5.1 Water Pumping Systems

As shown in Figure 14.2, pumping is generally required from source to WTP and again from WTP to clear water master balancing reservoir (MBR).

The water pumping systems can and need to be monitored for the parameter such as water level, pressure, incoming and outgoing flow, energy consumed by the pumping systems, water quality parameters, among others.

Monitoring and Control using SCADA

(i) Monitoring: Monitoring location and parameters are shown in Table 14.1.

Table 14.1: Monitoring location and parameters

Location	Parameter	Method	Per
River Source	Level	Ultrasonic/Radar Level Sensor	Take Off Point
Pump House	Level	Ultrasonic/Radar Level Sensor	Underground Reservoir
	Flow	Electromagnetic Flowmeter	Individual Delivery of the Pump
	Flow	Electromagnetic Flowmeter	Common Header from the Pump House
	Energy	Electronic Energy Meter	Common LT/HT Line to the Pump House
	Energy	Electronic Energy Meter	LT/HT Line feeding the individual Pumps at Pump House

(ii) Control: Control location and parameters are shown in Table 14.2.

Table 14.2: Control location and parameters

Location	Parameter	Method	Per
River Source	Flow	Electric Actuated Gate/Valves	Off Take Point
Pump House	Priming System	Electric Actuated Valves/Solenoid valves	Pump at Inlet line
	Flow	Electric Actuated Gate/Valves	Individual Delivery of the Pump

	Flow	Electric Actuated Gate/Valves	Common Header from the Pump House

'Per': suggests the location of installation,
All parameters must be available as 4–20 mA inputs to the monitoring and control system in real time for proper management of system.

14.5.2 Water Treatment Plants (WTP)

All water supply system will involve raw water being picked up from a source (river/check dam/canal, etc.). The source is generally a distance away from the town or the city which is being fed by the water supply system. C&I scheme for a WTP is shown in figure 14.3.

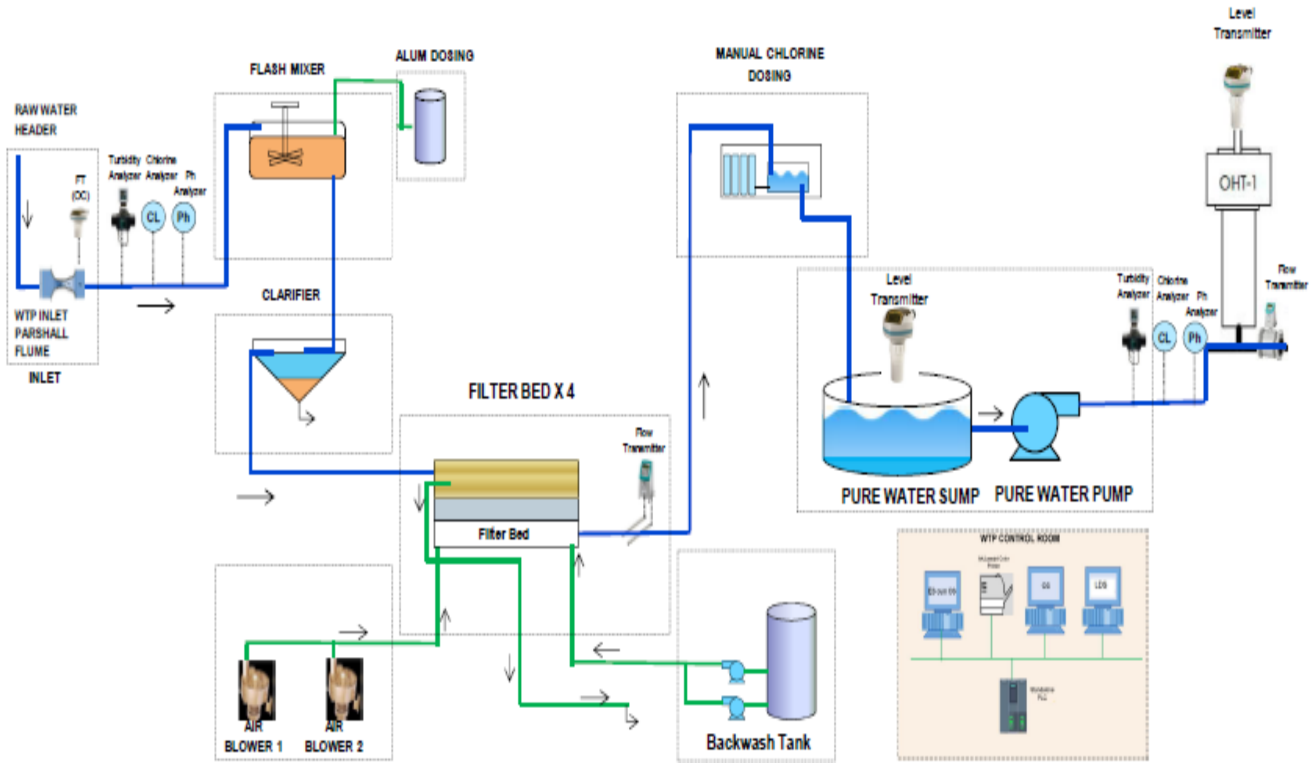


Figure 14.3: C&I scheme for WTP

The following table provides a list of parameters that can be considered for the purpose of real time and SCADA base monitoring.

Monitoring: The parameters to be monitored at each stage shall be considered as shown in Table 14.3.

Table 14.3: Monitoring location and parameters in WTP

Location	Parameter	Method	Per
Aerator	Flow	Electromagnetic Flowmeter	Aerator Inlet
Flash Mixer	Water Quality: pH and Turbidity	Online Sensors	Per Flash Mixer
Alum/PAC Dosing	Flowmeter of Alum/PAC	Electromagnetic Flowmeter	Per Dosing Pump
Clarifier	Water Quality: pH and Turbidity	Online Sensors	Per Clarifier Outlet

Location	Parameter	Method	Per
Filter Beds	Level	Ultrasonic Level Sensor	Filter Bed (multiple in case of multiple outlets in filter bed)
	Pressure	Differential Pressure Sensor	Filter Bed
Back Wash System	Level	Ultrasonic Level Sensor	ESR
	Pressure	Pressure Transmitter	At inlet line to the Filter Bed
Backwash Water Recirculation System	Level	Ultrasonic Level Sensor	Recirculation Sump
	Pressure	Pressure Transmitter	At outlet line from the pumping system
	Flow	Electromagnetic Flowmeter	At inlet to the recirculation sump/recirculation line to the pre-clarifier stage
Chlorine Dosing System	Residual Chlorine	Online Sensors	Per Pure Water UGR
Pure Water Pumping System	Pressure	Pressure Transmitter	At outlet line from the pumping system
	Level	Ultrasonic Level Sensor	Sump
	Flow	Electromagnetic Flowmeter	At outlet line from the pumping system

(ii) Control: The parameters to be controlled at each stage are shown in Table 14.4.

Table 14.4: Control location and parameters in WTP

Location	Parameter	Method	Per
Aerator		PLC based flow monitoring local panel	
Flash Mixer	Mixer Operations	PLC controlled local panel	Per Flash Mixer
Alum/PAC Dosing	Flow of Alum/ PAC	PLC controlled local panel operating Dosing Pump	Per Dosing Pump
Clarifier	Clarifier Operations	PLC controlled local panel	Per Clarifier
Filter Beds	Filter Bed Operations of Regular, Backwash and Drain Modes	PLC controlled local panel	Filter Bed
	Flow	Electric Actuated Gate/Valves	Individual Inlet/Outlet/Backwash

Location	Parameter	Method	Per
			and Drain of the Filter Bed
Back Wash System	Priming System	Electric Actuated Valves/Solenoid valves	Pump at Inlet line
	Flow	Electric Actuated Gate/Valves	Individual Delivery of the Pump
	Pressure	Pressure Transmitter	At outlet line from the pumping system
	Flow	Electromagnetic Flowmeter	At inlet to the recirculation sump/recirculation line to the pre-clarifier stage
Chlorine Dosing System	Residual Chlorine	Online Sensors	Per Pure Water UGR
Pure Water Pumping System	Pressure	Pressure Transmitter	At outlet line from the pumping system
	Level	Ultrasonic Level Sensor	Sump
	Flow	Electromagnetic Flowmeter	At outlet line from the pumping system

14.5.3 ESR/MBR/GSR

The ESR/MBR/GSR systems need to be monitored for the parameters such as water level, pressure, incoming and outgoing flow, water quality parameters among others. A typical inlet and outlet arrangement at service reservoir is shown in Figure 14.4. This arrangement is only for a sump/clear water reservoir pumping water to the ESR. However, for a gravity/pumping main supplying water to multi-ESRs, the pressure gauge as shown in Figure 14.4 will be replaced by a pressure transmitter which will be connected to RTU. In addition to this, the on/off sluice/butterfly valve shall be replaced by the dual solenoid FCV.

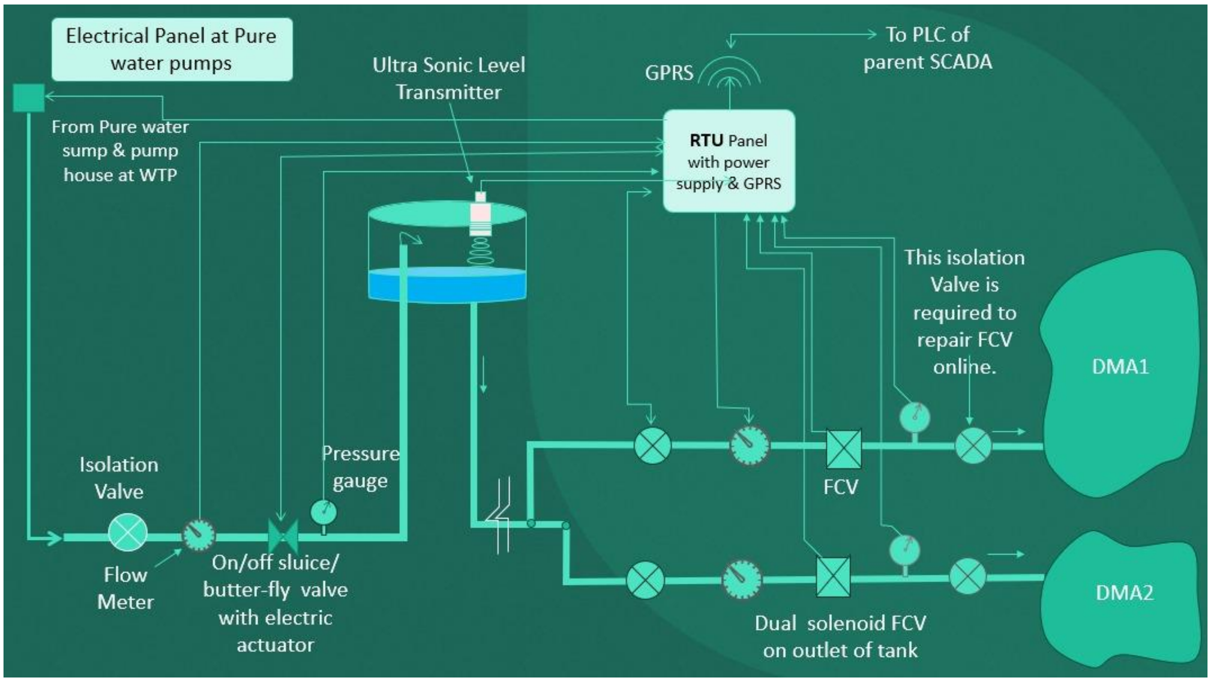


Figure 14.4: Typical inlet and outlet arrangement at service reservoir

Typical FCV is shown in Figure 14.5. FCVs are used to rationalise (less quantity in non-peak hours and more quantity at peak hours) distribution of water from ESRs to DMAs. Normally, with hydraulic model, the design of pipelines is so made that the DMAs can get water as per their demand. However, the tank practically gets empty (on sloppy side) when the flow starts. But with FCV, the flow can be regulated in such a way that the water will flow as per the actual demand in the DMA and there will be backward pressure available which will help to maintain water level in the service tank.

During pipe breakages or burst in the DMA, the FCV at the entry of DMA can close the flow. Here, the pressure transmitter shall send the signal and accordingly, the PLC shall control the FCV through its solenoids.

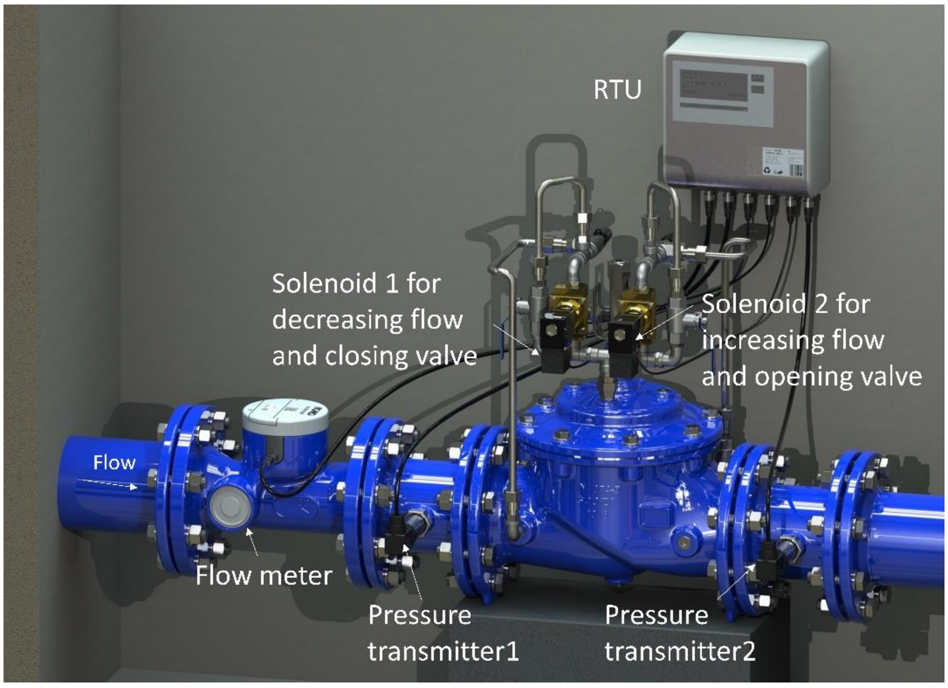


Figure 14.5: Typical FCV with solenoids, RTU, pressure transmitters and flowmeter.

In order to maintain the nodal pressure of 21 m, if VFD pump is installed on the outlet of the ESR, then FCV may not be necessary.

Monitoring and Control by IoT System

Monitoring: The parameters to be monitored are shown in Table 14.5.

Table 14.5: Monitoring location and parameters in ESR/ UGR/ pump house

Location	Parameter	Method	Per
Elevated Service Reservoir	Level	Ultrasonic/Radar Level Sensor	Reservoir
Pump House	Level	Ultrasonic/Radar Level Sensor	Underground Reservoir
	Flow	Electromagnetic Flowmeter	Individual Delivery of the Pump
	Pressure	Pressure Transmitter	Individual Delivery of the Pump
	Flow	Electromagnetic Flowmeter	Common Header from the Pump House
	Pressure	Pressure Transmitter	Common Delivery of the Pumps
	Energy	Electronic Energy Meter	Common LT/HT Line to the Pump House
	Energy	Electronic Energy Meter	LT/HT Line feeding the individual Pumps at Pump House

Control: The parameters to be controlled are shown in Table 14.6.

Table 14.6: Monitoring location and parameters in ESR/ UGR/ pump house

Location	Parameter	Method	Per
ES Reservoir/UG Reservoir	Flow	Electric Actuated Gate/Valves	Inlet, Bypass and Outlet
Pump House	Priming System	Electric Actuated Valves/Solenoid valves	Pump at Inlet line
	Flow	Electric Actuated Gate/Valves	Individual Delivery of the Pump
	Flow	Electric Actuated Gate/Valves	Common Header from the Pump House

'Per': suggests the location of installation,
All parameters must be available as 4–20mA inputs to the monitoring and control system in real time for proper management of system.

14.5.4 Key points in Distribution System

In addition to the above systems/subsystems, every water supply distribution system has some key notes where it is important to measure various parameters such as pressure, flow, water quality to ascertain that the water supply system is working within normal and allowed limits. Such key locations are at the entry of each DMA and the strategic spots such as highest and lowest ground elevations and farthest node in the operational zones. It becomes key to be able to get these feedbacks from predefined locations as they are litmus test for the proper functioning of the overall system.

Over a period of time, these key points may lose/gain importance for parameter measurement in the overall water supply system.

In addition to the pre identified key points as the water supply system expands and evolves over time, there may be a requirement to add or reduce key points in the water distribution set up.

The following sections deal with a broad outlook on the requirement of parameter monitoring in the various subsystems and gives an insight into what may be considered by the various utilities/boards of corporations for incorporation in their individual water supply system.

Monitoring: The parameters to be monitored are shown in Table 14.7.

Table 14.7: Monitoring location and parameters

Location	Parameter	Method	Per
Entry of DMA	Flow, pressure	Ultrasonic/Electromagnetic Flowmeter and Pressure Transmitter	Inlet
Strategic points such as highest and lowest elevations and farthest node in operational zone	Pressure	Pressure Transmitter	As per the topography
Strategic Points	Water Quality Parameters	Water Quality Analyser	As per the criteria given in Chapter 8 of the Part B manual

Control: The parameters to be controlled are shown in Table 14.8.

Table 14.8: Controlling parameters

Location	Parameter	Method	Per
Entry point of DMA and sub-DMA	Flow	Electric Actuated Gate/Valves	Key Point

'Per': suggests the location of installation
All parameters must be available as 4–20mA inputs to the monitoring and control system in real time for proper management of system.

The control systems and all the above subsystems would comprise of PLC/RTU-based control panels with accessories and relevant switchgear as per the system requirement that can be programmed to operate the control systems in a pre-programmed fashion. In addition, as required and as per site conditions, the PLC/RTU panels will have the capability to provide localised communication on technologies such as OFC, ethernet, wireless CAN, and on wireless technologies like GPRS, satellite communication, LoRaWAN, etc.

The control and monitoring elements should further be enabled to communicate the status of their individual locations to an IoT based centralised monitoring and control centre (CMS) that enables the water supply system to be monitored from one single location for the entire state/city/town.

Controls and instrumentation should be planned as per the projected requirement viz., appropriate plant size, complexity, and number of staff and their skills for each plant. To achieve this, the designer should develop a control philosophy that will enable the plant staff to effectively monitor and control the plant and major equipment, the water supply system.

14.6 SCADA and IoT Comparison in Water Distribution Systems

Main characteristics of a water distribution system are:

1. large areas coverage with seamless and very low-cost connectivity;
2. large data crunching;
3. real-time monitoring and data analytics;
4. historical data context;
5. making data available to other systems.

Table 14.9: Comparison of SCADA and IoT

SN	IoT/Digital Platform	SCADA
1.	Designed for web-based monitoring of multiple devices/processes	Designed for primary control of a process
2.	A web-based system suitable for cloud deployment	A standalone system with web interface
3.	ISA-95 level 3 system. Can connect to SCADA for data collection	ISA-95 level 2 system
4.	Main focus is various value-added features using collected data. Has capability of control functions like start/stop pump or open/close valve, etc. Latency is in seconds/minutes depending on network latency	Main focus is operations and real-time control. Time-critical controls with milliseconds latency are possible
5.	Multi-user system with different user roles and elaborate access right allocation	Multi-user system. Access rights allocation is limited
6.	Used by all user levels up to management	Mainly used by plant operators, engineers and supervisors
7.	Core features are dashboards, reports and analytics on data collected directly from plant/process. Also, notifications to users for real-time actions	Core features are monitoring and control of plant/process
8.	Can be integrated with other enterprise systems like billing system, GIS	Difficult to integrate with enterprise systems

14.7 District Metering Areas (DMA) or Sub-DMA Monitoring and Control

A District Metered Area (DMA) is a subzone within operational zone of a water distribution network that can be hydraulically isolated and for which water consumption is measured using water meters. Bulk flowmeters are installed at the entry points of the DMAs, and all user connections are properly metered for recording the consumption.

The main purpose of DMA is to identify and prioritise leak identification and repair programme by computing NRW values. Another important purpose of DMA is to rationally distribute the water according to the needs with equal pressure. A typical hydraulically discrete single DMA is shown in Figure 14.6.

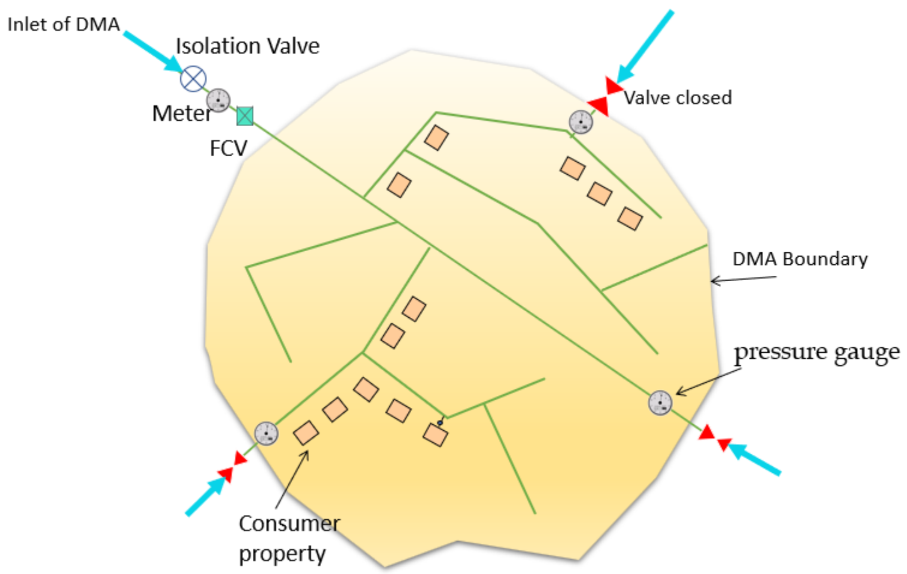


Figure 14.6: Typical hydraulically discrete single DMA

The isolation valve, meter, and the flow control valve (FCV), at the entry point of the DMA, should be connected to the SCADA/IoT.

14.7.1 Criteria for DMA

- Size of DMA (e.g. number of connections – generally between 500 and 3000 in plain areas)
- Number of valves that must be closed to isolate the DMA
- Number of flowmeters to measure inflows and outflows (the fewer meters required, the lower the establishment costs)
- Ground-level variations and, thus, pressures within the DMA (the flatter the area the more stable the pressures and the easier to establish pressure controls)
- Easily visible topographic features that can serve as boundaries for the DMA, such as rivers, drainage channels, railroads, highways, etc.

To divide a large open system into a series of DMAs, it is essential to close valves to isolate a certain area and install flowmeters. This process can affect the system's pressures, both within that particular DMA as well as its surrounding areas. The water utility, therefore, must ensure that the water supply to all customers is not compromised in terms of pressure and supply hours. The following aspects of the DMA are to be recorded and maintained by the engineer at site for effective management of water supply.

Table 14.10: Aspects of the DMA are to be recorded and maintained

S. No	Description	Remarks
1	Area of a DMA	
2	Source of water	
3	Point of inlets and outlets	
4	Total length of pipeline network as diameter-wise	
5	Age of pipeline network	
6	Total number of HSCs	
7	Total number of bulk connections	
8	Base line leakage levels	
9	Number of valves and its location	
10	Valves to be operated to deliver water for the entire DMA	
11	Supply hours and supply zones	
12	Low pressure region of the DMA	
13	Population of the DMA	

14.7.2 Boundary demarcation (Natural and artificial)

Using a calibrated hydraulic network model of the supply system to simulate possible DMA designs will enable analyses of system pressures and flows without affecting supply to customers. However, many water utilities/municipalities/water boards do not have existing calibrated hydraulic network models. The boundary demarcation shall factor in railway crossings/Nallah/river/NHAI crossings such that the DMA does not extend along these features. Also, the terrain within the DMA shall not be too much undulating. The most important factor governing the DMA demarcation is to ensure that the ELSR/OHT is located at a significantly higher elevation within the DMA.

14.7.3 DMA Isolation

DMAs in network areas should be easily isolated, i.e. areas with a separate supply zone. In establishing a DMA, the water utility should limit the number of inflows, which also helps to reduce the cost of flowmeter installation. To achieve this, it is necessary to close one or more boundary valves, which must remain shut permanently to ensure that any flow data accurately represents the total inflow for the DMA. Understanding district meter areas utility managers will ensure that all pipes into and out of the DMA are either closed or metered by performing an isolation test as follows:

1. Close all metered inlets.
2. Check whether the water pressure within the DMA drops to zero, since no water should now be able to enter the area.

If the pressure does not drop to zero, then it is likely that another pipe is allowing water into the area and therefore needs to be addressed.

Once the DMA has been established, it becomes an operational tool for monitoring and managing both of the major components of NRW, physical and commercial losses. The calculation for NRW within a DMA is defined as follows:

DMA NRW = Total DMA Inflow - Total DMA Consumption

After flowmeters are installed on all inlets to the DMA, the Total DMA Inflow can be measured using the increase in the totaliser, or the meter counter measuring the volume of water passing through the meter, for the calculation period. The total DMA consumption will depend on the customer meter coverage. If the DMA has a 100% domestic meter coverage, meaning all customers within the DMA have a meter, then the total DMA consumption can be calculated through a simple summation of all meter measurements for the calculation period.

The RTU Panel shall be installed with GSM-GPRS-LTE-NB LTE unit to transfer the field process and non-process parameters to the cloud server or centralised monitoring and control centre or web/mobile-based information retrieval system. Process parameters include:

- Real time – Upstream – Downstream – Pressure
- Real time – Flow rate
- Real time – Flow quantity
- Real time – Pressure – Flow control – Open/Close
- Real time – Pressure – Flow – Proportional control

14.7.4 Monitoring DMA inflows and pressures is used for calculation of:

- System input volume (SIV).
- Minimum night flow (MNF).
- Supply pressure at inlet point and at critical and average pressure point of the DMA.
- Online bottom-up water balance including leakage, water demand estimation.
- Detection of bursts events.
- Water loss performance Indicators: including leakage per service connections/per length of pipes, and the infrastructure leakage index (ILI).

14.7.5 DMA Operation may be controlled for:

- Setting supply pressure at DMA inlet point.
- Controlling isolating valves to control supply in case of failure or shortage or to manage supply for sub-DMAs in case of intermittent supply.

This information is used to score DMAs based on their situation, set targets, and prioritise activities. A typical District Metered Area schematic diagram is shown in Figure 14.7.

Pressure Control

Pressure reduction is highly efficient in reducing and controlling leakage. Pressure is usually modulated by installation of pressure reducing valve (PRV) at the DM inlet chamber to control the downstream pressure.

Three PRV control types are enabled:

1. Fixed pressure control: PRV is set to control one fixed downstream pressure.
2. Two-point control: PRV set point changes between day and night time.
3. Dynamic pressure control: Downstream pressure is controlled according to flow, time or pressure at remote point, typically the DMA critical point.

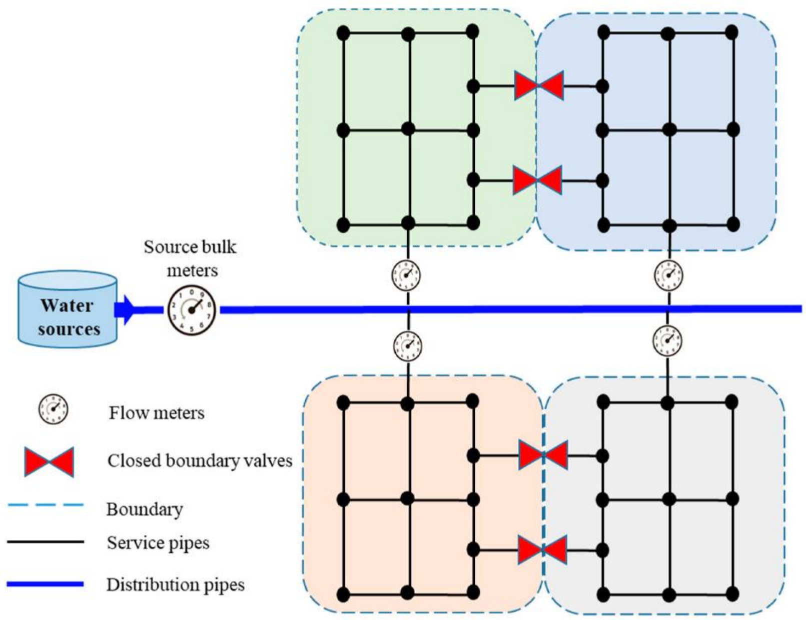


Figure 14.7: District Metered Area schematic diagram

14.8 Monitoring of NRW in 24×7 Water Supply System at DMA Level

14.8.1 Monitoring of NRW at DMA level and communication technologies (IoT)

The purpose of DMA formation is to monitor the water supply into DMA and measure the consumption of water at household level to monitor and control NRW in a gradual manner as explained in Chapter 2 of Planning, Investigations, Design, and Implementation of Part A Manual. This will also facilitate to conduct regular water audit and take measures to bring down the NRW to the desired level. In order to monitor the NRW at DMA level, the prerequisite is to provide flowmeter at the entry of DMA and 100% House service connections with water meters and proper communication technologies. The frequency of monitoring of NRW and accuracy depends on the type of water meters and communication technologies adopted in the 24×7 water supply distribution system at DMA level.

The different water meters with communication technologies for water meter data transmission to the servers to assess the NRW audit are discussed as under with merits and demerits.

14.8.1.1 Consumer Meter and Data collection

The specifications of the bulk and consumer metering determine the framework of meter reading methodology. There are four types of meter data transfer to collect and analyse the supply and consumption of water.

- MultiJet non-AMR meters
- AMR compatible meters
- AMR meters
- Smart water meters – electromagnetic and ultrasonic types

Meter Reading and Management requirements

- During DMA study, readings from all consumer water meters of selected DMA consumers are to be recorded in accordance with requirements.
- Develop a monitoring programme of random spot-checks to ensure the accuracy of the meters and the meter reading process to be carried out by the ULB.
- Develop and implement a plan which would ensure

  - All consumer meters are in working condition
  - All consumer meters are accurate,
  - All consumer meters are read,
  - All consumer meters are in suitable and easily approachable locations,
  - Problems related to unprotected and unsealed consumer meters are resolved,
- Develop and implement a programme to estimate consumption in circumstances where metering problems exist and provide advice as to methods to improve the meter reading process to ensure greater accuracy.
- Identify consumer meters which have not been read.

14.8.2 Type of Smart Meters available with Communication Technologies

There are various types of water meters that are being used in WS&D projects. They are classified based in principle of operation namely:

- **Mechanical type Water Meter – Single Jet, MultiJet**
  This type of water meter works on the principle of turbine rotation with respect to flow of water through the water meter. When the water passes through the flow tube, a small turbine inside the water meter rotates and magnetic pulse will be generated from the magnetic pickup installed in the turbine. The name single jet or MultiJet is based on the number of water jet inside the flow tube. In a single jet meter, water will hit the turbine in single nozzle or jet, whereas in MultiJet, the water will divide multiple jets and hit the turbine. MultiJet meters are more accurate and long-term use compared to single jet meters. These meters will not have any facility to communicate.

- **MultiJet Manual/Non-AMR Water Meter**
  These water meters will record the reading of volume (m^3) in the form of mechanical registers or electronic registers. These meters will not have any facility to communicate. Hence, the meter reader has to go personally to the meter location, open, read, and enter the reading manually in the reading register (paper or electronically). This is mainly for mechanical meters which do not have the remote reading facilities.

- **MultiJet AMR Compatible Water Meter**
  These water meters are like manual water meters except with an additional provision of external communication provision. These meters can have the facility of pulse output or serial output, which can be sensed by an external device and converted into electronic media and to be communicated through radio/GSM/GPRS networks. Note that these mechanical types of meters have only the facility of integrating the conversion and communication the reading.

- **Electromagnetic Type Water Meter**
  Electromagnetic water meter functions on the principle of electromagnetism, called Faraday's induction law, to measure the velocity of the water passing through it. In an electromagnetic meter, a magnetic field is created across the pipe. When water, which is an electrical conductor, moves through the magnetic field, a voltage is induced which is detected by electrodes in the body of the meter. The voltage is directly proportional to the flow velocity, which is used to calculate the volume. Electromagnetic water meters are accurate within their measuring range. Normally, DN15 to DN40 are referred to as domestic electromagnetic water meter, while meters from DN50 to DN300 are referred to as bulk electromagnetic meter.

- **Ultrasonic Type Water Meter**
  Ultrasonic water meters utilise the properties and behaviour of sound waves passing through moving water. The ultrasonic water meters are designed on the transit time principle. A transit time ultrasonic meter has two sound transducers mounted on opposite sides of the pipe at an

angle to the flow. Each of these sound transducers will, in turn, transmit an ultrasound signal to the other transducer. The differences in the transit times of the signals determine the velocity, and the volume is derived from the velocity. The measuring principle is called 'bidirectional ultrasound technique based on the transit time method', which is a proven, long-term, stable, and accurate measuring principle.

- **Automated Meter Reading (AMR) Water Meter**
  Mechanical water meters are an updated version of mechanical AMR compatible water meters. These meters are fitted with an electronic device, which can pick up the reading from the water meter. For electromagnetic and ultrasonic water meter, AMR facility is in-built, making them ‘smart meters’.
  In AMR, meter readings are automatically collected through radio frequency media without manual intervention. This metering data is then transferred to a central computer through a cellular network for analysis and billing purposes. Readings from AMR can be obtained by a simple walk-by or drive-by method, where the meter reader either walks or drives down the street while automatically downloading the meter data.
  - **Walk-By Method:**
    The meter reader has to carry a meter reading device and walk through the service area (Figure 14.8). The meter reading device will automatically pick up the water meter readings of water meters around. These reading devices also have software facilities to identify the meters which are not responding or are faulty. Once the meter reader travels through the designated route, the readings are transferred to a central server via the cloud using GSM/GPRS mode.
  - **Drive-By Method:**
    The procedure is as same as the walk-by method, except that the reading device shall be mounted on a vehicle (bike or van) with high-gain antennas mounted on top of the vehicle (Figure 14.8). The reading, once taken, will be transferred to a central server through GSM/GPRS mode.

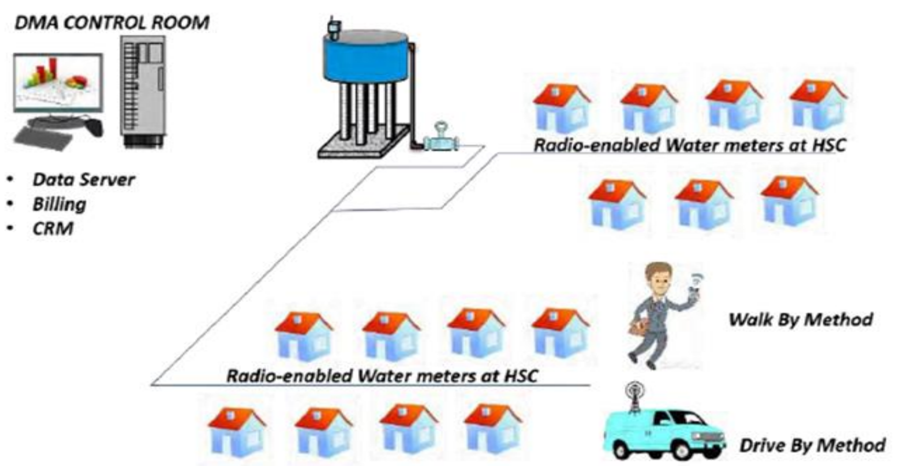


Figure 14.8: Walk-by Method and Drive-By Method

- **Automated Metering Infrastructure (AMI) Meter:**
  Advanced Metering Infrastructure (AMI) is the collective term to describe the whole infrastructure from smart meter to two-way communication network to control centre equipment and all the applications that enable the gathering and transfer of energy usage information in near real time. This is an AMI and differs from automatic meter reading (AMR) in that it enables frequent reading

of the meters. Communications from the smart water meter to the network is always through wireless communication.

AMI refers to an infrastructure of fixed network for monitoring and collecting the reading from water meters. Currently, there are two common infrastructure facilities available for data transfer through AMI.

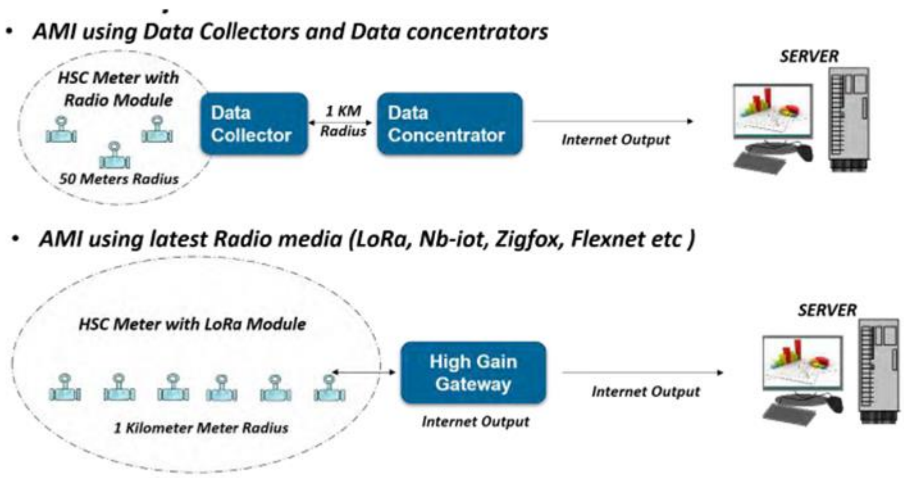


Figure 14.9: Two common AMI infrastructure facilities

**Range depends upon radio performance with various obstacle, LOS, data packet size.*

The comparison between different water metering and communication technologies is provided in Table 14.11.

Table 14.11: Comparison in Between Different types of smart meters

Characteristics	Mechanical Meters	Mechanical Meters	Mechanical Meters	Ultrasonic Water Meters	Electromagnetic Water Meters AMI
	Non-AMR	AMR	Advanced AMR	AMI	
Approx Cost per meter	X	3X	3.5X	4.5X (Composite Body) 6X (Brass/Bronze Body)	4.5X (Composite Body) 6X (Brass/Bronze Body)
Approx Lifetime of product (Considering Brass/Copper Alloy Body and Register version)	5 years	5 years	5 Years	10–12 Years	10–12 years
Accuracy	± 5%	± 5%	± 5%	± 2%	± 2%
Battery Life (Based on One Communication per Month)	NA	5 Years	5 Years	12 Years	12 Years
Battery Life	NA	5–6 Years	5-6 Years	10 Years	10 Years

Characteristics	Mechanical Meters	Mechanical Meters	Mechanical Meters	Ultrasonic Water Meters	Electromagnetic Water Meters AMI
	Non-AMR	AMR	Advanced AMR	AMI	
(Based on One Communication per Day)					
Battery Life (Based on One Communication per hour)	NA	4–5 Months	5 Months	10 Years	10 Years
Consumption Index Storage Frequency	NA	Daily Index available while fetching data through AMR	Hourly Index available	Daily Index and Hourly Index	Daily and Hourly Index
Consumption index recording capability during Minimum Night Flow (MNF) study	NA	Daily Index recording can be converted to Hourly but affects battery	Yes (15 mins index recording possible for 4 hours per day)	Hourly Index recording can be possible	Hourly Index recording can be possible
Interoperability (Capability to convert AMR to AMI and vice versa)	NA	Not possible	Yes	Yes	Yes
Dynamic Operating Range (R=Q3/Q1) As per MID certification	Up to R50	Up to R 80	Up to R 80	Up to R 400	Up to R 400
Minimum accountability (15 MM)	50 Litre/Hour	30 Litre/Hour	30 Litre/Hour	6 Litre/Hour	6 Litre/Hour
AMR Dataset capability to do online high-resolution water balance and leak estimation	NA	Daily Water Balance Possible, Leak estimation not possible	Hourly Water Balance Possible and Leak Estimation Possible	Hourly Water Balance Possible and Leak Estimation Possible	Hourly Water Balance Possible and Leak Estimation Possible

Characteristics	**Mechanical Meters**	**Mechanical Meters**	**Mechanical Meters**	**Ultrasonic Water Meters**	**Electromagnetic Water Meters**
	Non-AMR	**AMR**	**Advanced AMR**	**AMI**	**AMI**
Reverse Flow Detection through AMR (Reverse flow detection dataset can be used to detect probable sources of contamination in water network)	No	Yes	Yes	Yes	Yes
Pressure Rating (in Bar)	PN16	PN16	PN16	PN16	PN16
Environmental Consideration (Recyclability of Product at end of life)	Yes	Yes	Yes	No (Composite Body) Yes (Brass/Bronze Body)	No (Composite Body) Yes (Brass/Bronze Body)
Communication Technology:					
Available options	No	Radio Frequency (RF)	RF, LoRaWAN, NB-IoT	RF, LoRaWAN, NB-IoT	RF, LoRaWAN, NB-IoT
Metering Index Technology	Analogue	Analogue	Analogue	Digital	Digital
Pulse Reader to convert data from Analogue to Digital	NA	Required	Required	Not Required	Not Required
Communication Module	NA	Required	Required	Required	Required
RF Technology	NA	Drive-by/Walk-by	Drive-by/Walk-by	Drive-by/Walk-by	Drive-by/Walk-by
LoRa/LoRa WAN	NA	Drive-by/Walk-by	Drive-by/Walk-By/Fixed Private Network	Drive-by/Walk-by/ Fixed Private Network	Drive-by/Walk-by/Fixed Private Network

Characteristics	Mechanical Meters	Mechanical Meters	Mechanical Meters	Ultrasonic Water Meters	Electromagnetic Water Meters AMI
	Non-AMR	AMR	Advanced AMR	AMI	
NB-IoT	NA	NA	Fixed Network provided by Cellular Company	Fixed Network provided by Cellular Company	Fixed Network provided by Cellular Company
Daily Reading/NRW Audit	NA	NA	Possible	Possible	Possible
Changing of Communication Interval	NA	NA	Possible	Possible	Possible
Two-way Communication	NA	Not possible	Possible	Possible	Possible

** The battery consumption shall be depended on the communication intervals, so it is advised to minimise the same if not required, to ensure the life of battery.

The key to an effective NRW reduction is to develop a better understanding of the causes for NRW and its components. This is to be established as a part of the water audit through a diagnostic approach by establishing network characteristics and understanding operation and management practices.

14.9 Sensor Systems

14.9.1 Mechanical

Mechanical system can be used to measure, record, and trend variations in process parameters, such as pressure, flow, or temperature. In most cases, the instrument will convert mechanical displacement or movement of its sensor into a corresponding proportional movement of a pen or gauge.

Control of mechanical systems involves using:

(i) Sensors to monitor:
1. the input;
2. the mechanisms in the process part of the system;
3. the output motion and force.

(ii) Feeding the information to:
1. analogue or digital displays so that a human operator may act on the information; and/or
2. controlling devices that act on the input and/or process parts of the system to achieve a desired output.

14.9.2 Pneumatic

A pneumatic system is a system that uses compressed air to transmit and control energy. Pneumatic components are resistant to continuous loads and maintenance-free over their entire service life. They are very easy to install and are cheaper than comparable electrical solutions, particularly when implementing complete system solutions. Pneumatic components also have benefits when it comes to safety. Compressed air continues to be available even during a power failure. An air reservoir is always available along with a compressor for generation and preparation purposes. Apart from end-position sensing and monitoring the compressed air supply, it doesn't need to be monitored and checked. It follows the 'fit and forget' principle.

Pneumatic actuators only require electricity for the control and generation of compressed air; the movement itself is triggered by the compressed air. Whereas electric actuators require gearboxes, which are responsible for most of the power losses as well as electrical heat losses, pneumatic actuators act directly on the shut-off device. They only require a piston and drive shaft to convert the 'linear' compressed air force into a swivel motion.

14.9.3 Electrical

Electrical instrumentation is about the design, realisation, and use of electric or electronic systems for the measurement of electrical and non-electrical quantities. Strongly related fields are measurement science and data acquisition. A control system that uses an electric current, either direct current (DC) or current shuttle (AC) as a source of supply. Electrical systems require:

(i) electricity (DC) or (AC);
(ii) input elements (switches, sensors, transducer, valves, electronic components, etc.);
(iii) output elements (motor, lights, etc.);
(iv) extension cable.

This system of instrumentation and control is simple and widely used.

14.9.4 Electro-pneumatic

Electro-pneumatic control consists of electrical control systems operating pneumatic power systems. In this, solenoid valves are used as interface between the electrical and pneumatic systems. Devices like limit switches and proximity sensors are used as feedback elements. Electro-pneumatic control integrates pneumatic and electrical technologies and is more widely used for large applications. In electro-pneumatics, the signal medium is the electrical signal either AC or DC source is used. Working medium is compressed air. Operating voltages from around 12 V to 220 V are often used. The final control valve is activated by solenoid actuation.

In electro-pneumatic controls, mainly, three important steps are involved:

(i) Signal input devices – Signal generation such as switches and contactor, various types of contact and proximity sensors;
(ii) Signal processing – Use of combination of contactors of relay or using programmable logic controllers;
(iii) Signal outputs – Outputs obtained after processing are used for activation of solenoids, indicators or audible alarms.

Seven basic electrical devices commonly used in the control of fluid power systems are:

(i) manually actuated push button switches;
(ii) limit switches;
(iii) pressure switches;
(iv) solenoids;
(v) relays;
(vi) timers;
(vii) temperature switches.

Other devices used in electro-pneumatics are:

(i) proximity sensors;
(ii) electric counters.

The greatest advantage of electro-pneumatic is the integration of various types of proximity sensors (electrical) and PLC (Programmable Logic Controller) for a very effective control. As the signal speed with electrical signal can be much higher, cycle time can be reduced, and signal can be conveyed over long distances.

14.9.5 Hydro-pneumatics

The hydro-pneumatic system provides water automatically at defined pressure and quantities at many locations from a single pumping system. This system operates on variety of differential pressures. It operates automatically by sensing the pressure within the preset pressure range. Hydro-pneumatic systems are used to supply water at designed pressure and quantity from a single source.

14.9.6 Level Measurement

The level measurement is categorised into the top-down and bottom-up level, the top-down measurement is not susceptible to leakage, or it can be considered as less leakage process. The top-down measurement could make a contact with the fluid or sometimes it won't, while the bottom-up type makes contact with the process fluid.

The level measuring instruments are suitable to measure the level of water and it could be done directly or indirectly which would be done according to the application. A level measuring instrument would act as an indicator that shows the level of the water and this information would be carried in the form of AC signals, and this is done for the control purpose. The level switches would monitor if the liquid is at a high level or low level according to the set point.

The direct level measurement is an easy process. This measurement is based on physical principles such as fluid motions, floats, and thermal properties. There are many types of level measurements under this category including sight glass, float-operated, dipstick, dip rods, and lead lines. So, basically, the direct level measurement is by defining the position of the interface, and the density of the measured material is not a constraint.

The indirect level measurement is done by measuring other quantities like volume. So, this method measures level by determining some other parameters such as pressure, weight, or temperature.

14.9.7 Essential instruments

i. Ultrasonic Tank Level Sensors

Ultrasonic level sensors measure the distance between the transducer and the surface. Using the time required for an ultrasound pulse to travel from a transducer to the fluid surface and back (TOF). These sensors work without the need to touch the medium being measured. Ultrasonic sensors are ideal for difficult liquids such as wastewater. Ultrasonic level sensors are also ideal for continuous level measurement. While float switches and other sensors are adept at measuring when liquid levels are above or below a certain point. The nature of ultrasonic level measurement is such that levels can be sensed and displayed in real time.

ii. Hydrostatic level transmitters

These transmitters help in determining fluid contents of a container by measuring the pressure of the resting body of the fluid within it. The greater the force of liquid, the greater the volume of fluid. The measured static pressure of a liquid is proportional to the height of the liquid. And the static pressure is converted into an electrical signal. The exquisite structure, simple adjustment, and flexible installation methods provide convenience to use.

iii. Radar Level Sensors

Radar level transmitters, also called radar level gauges which can undertake a Non-contact continuous level measurement in liquids and solids with free space radar sensors. Whereas, Non-contacting radar, based on microwave technology, detects only surfaces that reflect energy. These transmitters work on the principle of a radar by using radio wave emissions. Mounted at the top of a tank filled with liquid, these transmitters send a radar signal into the liquid and receives a reflection of the signal. The transmitters then analyse the current fill level of the tank based on the time taken by the transmitted signal to return.

iv. Capacitance Level Transmitters

Capacitance level detectors, also known as capacitance level transmitters. Capacitance level transmitter offers continuous and point level detection in liquids with capacitance probes. These transmitters use liquid stored in a tank or container as a dielectric medium between two or more electrodes. The energy capacity of the capacitor circuit increases when there is more liquid and decreases if there is less liquid. Measuring the variations in the capacitance value, capacitance level transmitters calculate the level of the tank.

v. Float level transmitters

Float level gauge is mainly composed of magnetic float, sensor, and transmitter. The magnetic float ball changes with the liquid level and floats up and down along the catheter. The magnetic steel in the float ball attracts the reed switch at the corresponding position in the sensor, which changes the total resistance (or voltage) of the sensor. The transmitter then converts the changed resistance (or voltage) signal into a current signal output.

14.10 Flow Measurement

Flowmeter is a device that can be used to measure the amount of liquid that passes through it. It is an instrument which is used to conduct the flow measurement. Flowmeters measure the flow by two methods, some flowmeters measure the flow as the amount of the liquid passes through the flowmeter during a period of time. While other flowmeters measure the flow by measuring the total amount of fluid that has passed through the flowmeter.

Flowmeter consists of devices such as transducer and transmitter. The transducer is responsible for sensing the fluid passing through the primary device, while the transmitter receives a signal from the transducer and converts it into a usable flow signal. So, a flowmeter can be considered as the combination of these physical devices.

Flowmeters can be classified into differential pressure meters, variable area meters, positive displacement meters, magnetic, turbine, ultrasonic, and vortex. In differential pressure meters, a restriction is used to create a pressure drop. So, the flow rate can be determined by measuring the pressure drop across the restriction. In positive displacement meters, the measuring process takes place by precision-fitted rotors as flow measuring elements. The selection of flowmeter depends upon flow conditions and flow range, process conditions like pressure and temperature, and size of pipe.

14.10.1 Filter Flow Control

An operational control system, including media configuration, underdrain system, and backwash process, is an important consideration because it determines how water flow is controlled through the filter. Four basic types of operational control systems are used in gravity filtration, with some variances from plant to plant.

(i) **Effluent rate of flow:** This type of system controls the rate of treatment through a cell on the filter's discharge. Basic components include a flow-sensing device, rate controller, and modulating valve. A wide variety of flow-sensing devices may be used, including direct reading meter systems and indirect reading systems such as venturi or orifice plates. The rate control design also varies considerably, depending on the flow-sensing device and modulating valve design. Signals from the flow sensor, rate controller, and valve may be pneumatic, electric, mechanical, or a combination of these.

(ii) **Influent flow splitting:** An influent flow splitting filter system has an adjustable weir positioned at the entrance of each filter cell. The influent weir is the system's main component. A feed pipe or flume common to all filter cells carries the water to the individual weirs. The inlet weirs are adjustable so each can be positioned at the same elevation to obtain uniform flow splitting. Ancillary equipment often includes an influent control valve and an effluent hydraulic control point to maintain a minimum water level in a filter cell. The influent control valve is used to stop incoming flow if the cell needs to be removed from service, such as during a backwash event. The effluent hydraulic control point may consist of a downstream weir or upturned loop in the effluent pipe.

(iii) **Constant level:** This type of filter control uses a level-sensing device in each filter cell to communicate with a control valve in the effluent line. The communication method can take various forms, including mechanical linkage, pneumatic pressure, electrical signal, or a combination of these. A variety of level-controlling devices may be used. The most common is a butterfly valve equipped with an appropriate actuator to position the valve disc. Valve size selection is critical for operation, and selection should be made such that a small change in disc position doesn't drastically change the flow through the valve.

(iv) **Declining rate:** Declining rate filtration is one of the oldest and simplest methods of filter control. It is used with multiple filter cells – the greater number of cells, the better the performance. The system's main component is the filter media. To limit the maximum rate of filtration through a cell, a flow restriction may be placed in the effluent pipe or, in some special cases, on the influent side. The flow restriction may simply be an orifice plate. This flow restriction is used to limit the initial flow passing through the filter – essentially a speed limit for the filter. A two-position control valve may also be used where the valve is partially closed directly after a backwash to restrict flow of the cleanest cell.

14.10.2 Filter Flow Control Valve

All filters have either five or six valves associated with the filter bay, depending on the type of backwash procedure. The control valves associated with a filter are influent, effluent, drain and backwash. The fifth valve is either an agitator water supply valve or an air valve, depending on whether a surface agitator wash system or an air/water system is used.

In the filtration mode, treated water enters the filter box through influent piping or channel and fills the box. Water flow is down through the media, through the underdrain support system, and out through the effluent piping where flow is measured and controlled by a rate of flow controller. The rate of flow controller is commonly a venturi meter, a flow transmitter and a filter effluent valve. As the loss of head across the filter increases, the filter effluent valve must modulate to a more open position to maintain the set point of the flow. Filtered water flows through the rate of flow controller to the clear well. The filter media used in a filter is commonly thought to trap the suspended materials. However, filtration is a combination of both physical and chemical mechanisms. As water passes through the media, large particles are trapped, and suspended particles become stuck to the surface of the grains of filter media and to other particles previously adsorbed by the media. Filter run time is dependent on how many particles are present in the water supply to the filter, and the initial cleanliness of the filter following the backwash sequence. Run times can vary from as short as 15 hours or less to over 100 hours. In the filtration mode, grains of filter media tend to stick together forming mud balls because of sticky floc. Unless backwashing removes this material, mud balls can sink into the media and clog the filter. As the filtration process a progress, the filter media becomes more restrictive to flow, causing an increased head loss through the media. If allowed to progress too far, negative pressures can be produced in the effluent piping that can cause air binding. A loss of head transmitter is a transmitter that senses the differential pressure between the head of water in the filter bay and the pressure in the effluent piping. In an automatic backwash system, high loss of head is one of the signals used to alarm or start a backwash sequence.

Automatic filter backwash control provides a consistent process of cleaning, and in the case of multimedia filters, proper re-stratification of the media. Air-water backwash control is extremely critical. An automatic backwash control system provides all interlocks to control the pumps, blowers, and valves to provide a safe and reliable operation. In some cases, the automatic control system is designed to adjust the backwash flow rate based on temperature or viscosity of the water and terminate the backwash sequence based on time or the turbidity of the backwash water. In the automatic mode, all sequencing is done by the control system. The backwash procedure is started automatically after the control system receives a high loss of head alarm, a high turbidity alarm, or a preset filter run time has been reached.

14.10.3 Rate of Flow of Chemicals

(i) **Liquid Chemical Feed**

The chemical dose rate should be flow paced to the plant flow in the part of the process that the chemical is to be injected into. Two methods are typically used to achieve this: metering pump, or flowmeter and flow control valve on the chemical feed. In metering pump, positive displacement type (diaphragm, peristaltic or progressive cavity) pump should be used. The output of the pump is directly controlled by a 4–20 mA signal from the flow transmitter on the plant flowmeter. On plant shutdown, the flowmeter (usually the raw water flowmeter) will signal the metering pump to stop and a solenoid on the dilution water to close. A load cell or pressure (level) transmitter on the chemical storage tank should provide warning signals when chemical supply is low and should have alarm and initiate plant shutdown when the level becomes critically low (low-low level). In case of flowmeter control, the chemical feed rate is controlled by an in-line flow control valve. A PLC receives a 4–20 mA signal from the flow transmitter on the plant flowmeter. Using this dose rate set point, the controller will look at the flowrate on the chemical feed flow transmitter and signals the in-line flow control valve to a position that will control the feed rate to the established set point. On plant shutdown, the controller will signal the in-line control valve to close. Depending on the range of feed rates, multiple flowmeters and control valves may be required. As with the metering pump system, low and low-low level alarms/shutdown should be provided on the chemical storage tank.

(ii) **Dry Chemical Feed**

Dry chemical feed systems typically include a packaged bulk storage combination feeder and mixer. The feeder can be gravimetric or volumetric and will be controlled by a 4–20 mA signal from the flow transmitter on the plant flowmeter. The chemical feeder discharges to a dissolving tank where it is mixed with plant service water to form a solution (slurry) suitable for dosing. Plant service water flowrate is manually set and monitored by flow indicator. The rate needs to be suitable for the range of anticipated chemical feeds and its solubility; the rate may need to be manually adjusted seasonally. For a specific plant service water flowrate, the variation in chemical feed rate creates a corresponding variation in solution strength fed to the process water. The solution is fed via a hydraulic injector (if into a pipeline under pressure), or directly by gravity into open channels or tanks. On plant shutdown a signal from the raw water flowmeter will call for the dry feeder to stop and a solenoid valve on the plant service water feed to the dissolving tank to close. If an injector is used, a solenoid valve on the plant service water will also close.

14.11 Pressure Measurement

Pressure may be simply indicated on a gauge or transmitted by a transmitter. Pressure gauges are available to read both differential and single-ended pressure. By far, the most common measurements are single-ended, although differential gauges are used to read head loss on water and air filters.

(i) Where a pressure gauge is reading a pressure produced by a pump (normally required) the gauge should be protected from vibration by filling it with either silicone liquid or glycerine. Silicone should be used if the ambient temperature will fall below -30 °C.
(ii) The range of the gauge should be chosen so that it will normally operate at one-half to two-thirds of scale at normal design pressure; the gauge should not be operated full-time near the top end of the scale. This will provide some safety margin on overpressure as well as prolonging the life of the gauge.
(iii) The gauge should be installed where the lens will not get damaged and where it can be read easily. Choose a gauge with a top-mounted stem where it will be installed near the ceiling so that the dial will read right-side up.
(iv) For water applications (both raw water and treated), a bronze or 316 stainless steel bourdon tube mechanism should be used. For applications on chemical lines, the manufacturer should be consulted for compatibility between the process and the gauge material.
(v) On corrosive liquids and processes containing solids, or where the gauge material is not compatible with the process, an isolating diaphragm should be used between the process sense line and the gauge to protect the gauge.
(vi) A block and bleed valve should be installed between the process and the gauge, or between the process and the diaphragm, to take the gauge out of service.

Pressure transmitters are available to read either differential or single-ended pressures. The single-ended type may read either gauge pressure (the pressure relative to the atmosphere) or absolute pressure (relative to a vacuum). Absolute pressure measurements are not common. Differential measurements are commonly used to determine when a filter needs washing. The range of the transmitter should be chosen so that it will normally operate at one-half to two-thirds of scale at normal design pressure; the transmitter should not be operated full-time near the top end of the scale. This will provide some safety margin on overpressure as well as prolonging the life of the sense element in the transmitter. For water applications (both raw water and treated), a bronze or 316 stainless steel sensing diaphragm should be used. A block and bleed valve should be used between the process and the transmitter, or between the process and the isolating diaphragm, to take the transmitter out of service, and to facilitate calibration. Where a pressure gauge is not installed on the same line as the transmitter, an integral display should be provided on the transmitter for local indication.

14.12 Water Quality

A set of automatic measuring, storing and display system of water quality helps in resolving the limitations in manual monitoring. A sensor is an ideal detecting device to solve these problems. It can convert non-power information into electrical signals. It has many special advantages such as good selectivity, high sensitivity, fast response speed, and so on. Water treatment and distribution process with sampling points is shown in figure 14.10.

The general precautions in the Bureau of Indian Standards, i.e. IS-3025/1622 and/or Standard Methods for the Examination of Water and Wastewater – latest edition [Published jointly by American

Public Health Association (APHA), and American Society for Testing and Materials (ASTM)] shall be referred to for detailed information on sampling and testing procedures.

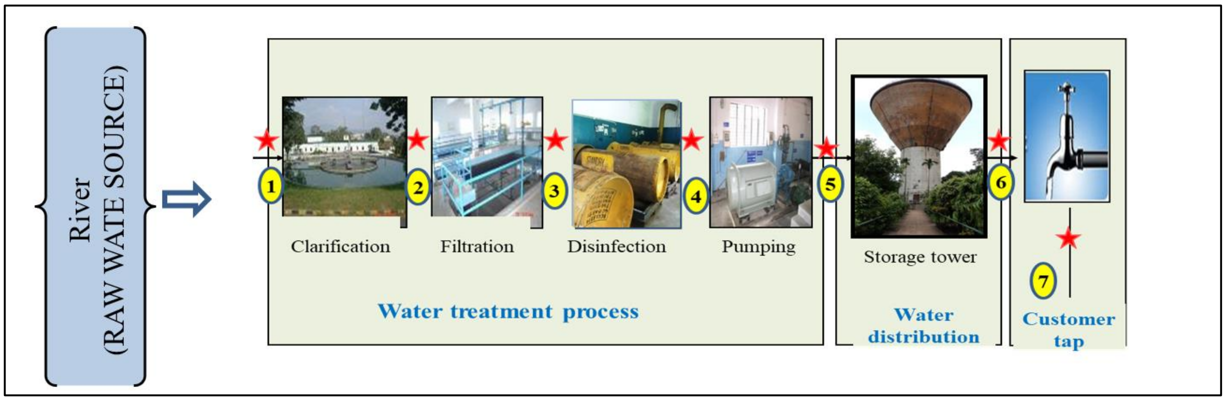


Figure 14.10: Water treatment and distribution process with sampling points

Sampling points and sampling schedule is shown in Table 14.12.

Table 14.12: Sampling points and sampling schedule

Sampling points	Sampling Schedule	Test Analysis	Critical Parameters for Process efficiency
1	Daily	Jar Test, Turbidity, pH, Colour, Coliform, Iron	pH 7.0 – 8.0, MPN of Coliform/100 ml ≤ 160000
2	Daily	Turbidity, pH, Colour, Coli form	Turbidity 2 – 6 N.T.U, pH 7.0 – 7.8, Coliform - ≤1600
3	Daily	Turbidity, pH, Colour, Coliform	Turbidity ≤ N.T.U, pH 7.0–7.6, Colour ≤5 Hazen, Coliform - ≤500 CFU, Aluminium ≤0.2 mg/L, Iron ≤0.3 mg/L
4,5,6 and 7	Daily	Turbidity, Free Chlorine, pH, Colour, Coliform, Aluminium, Iron	Turbidity ≤1 N.T.U, Free Chlorine 0.7 – 0.9 mg/L (WTP), ≥0.2 ppm (consumer tap) pH 7.0–7.6, Colour ≤ 5 Hazen, Coliform – absent, Aluminium ≤ 0.2 mg/L, Iron ≤ 0.3 mg/L

14.13 Quality Sensors

The most widely used water quality sensors in monitoring installations are temperature, specific conductance, dissolved oxygen (DO), pH, and turbidity. Sensors also are available to measure oxidation-reduction potential, water level, depth, salinity, ammonia, nitrate, chloride, and chlorophyll.

The sensors needed to measure these properties are available as single instruments or in various combinations. A group of sensors configured together commonly is referred to as a sonde, which typically has a single recording unit or electronic data logger to record the output from the multiple sensors.

Types of Monitor Configuration

In general, three types of configurations are used for water quality monitors:

1) The flow through monitoring system generally has a pump that delivers water to the sensor(s) mounted in a shelter;
2) The sensors are placed in situ (immersing a field measurement sensor directly into the water); and
3) The self-contained sensor and recording system that requires no external power and is placed in situ. Each configuration has advantages and disadvantages in relation to site location and data quality objectives.

14.14 Optional Instrumentation and Controls

14.14.1 Level

A level switch can be described as a device that can sense the level of a liquid in a process tank. A level switch is also used to control valves and pumps so that it can maintain the fluid level at a set value and also that it can prevent the tanks from overfilling. So, we can see a level switch in measurement and control applications, level switch can indicate the high and low levels in process tanks. There are many level switches, and the most common types of level switches are thermal, inductive, ultrasonic, etc.

14.14.2 Flow

Flow switches are used to detect the flow of the fluid, these switches are used to determine if the required flow is available or not. So, if the required flow is not available, then it could damage the system. There are different types of flow switches, and we can choose the flow switches according to the line size.

14.14.3 Pressure Switch Applications

Pressure switch is a device that monitors pressure and provides an output when a set pressure is reached. The specific pressure that opens and closes the switch is called the set point. Pressure switches are commonly associated with well pumps.

Pressure switches are comprised of several components including an adjustment screw, diaphragm, lever, and contacts. The adjustment screw sets the spring pressure. This can be adjusted to change the on and off pressure range that the switch operates at. The standard range of a pressure switch is usually located on the unit itself. It relies on water pressure to do all the work. The change in pressure that activates the switch is provided via the water from the well. The pressure moves up through the diaphragm which presses against a piston and spring, which in turn opens or closes the contacts. Open contacts located within the switch, closes when pressure drops. This completes an electrical circuit, which in turn, activates the pump. When the set pressure is reached, this allows the contacts to open again which turns off the pump. It is used to operate as a safety valve, which vents out pressure when the pressure exceeds the upper limit.

14.14.4 Filter Console

Filter control console provides continuous and discrete controls that are necessary for a typical surface or bulk filter in a WTP. This device can be used for standalone operation or it can be integrated into a plant-wide SCADA network. The PLC in the system eliminates the need for items such as timers, relays, and PID controllers. It also provides documented configuration that can be adapted by users to suit the unique needs for a given situation.

14.14.5 Clarifier Desludging

The control system associated with the clarifier consists of the mobile bottom valve hydraulically movable in the bowl; the control water feed, the control water valve system with the solenoid and manually controlled valves for automatic and manual control, and the programme control system. The control system is operated in normal operation by a preselected desludging programme. Any required full desludge operation can be controlled semi- automatically by the valve control cabinet. In the case of current failure or other possible defects, putting the control cabinet or the electrical control valves out of operation, the separation process can be brought to an end by operating the manually operated valves.

14.15 Instrument cum Control Panel

These panels are useful to monitor and control process parameters like temperature, pressure, flow, etc., and also to control the power to the final driver like motor pumps, heaters, etc. Various interlocks and safety audio visual alarms are provided through PLCs. Total process simulation can be shown on the mimic. Starters for motors, heaters, pumps are provided in the panel.

14.16 Online Measurement Instrumentation

14.16.1 Level Measurement

Whenever water parameters are considered, both water quantity and water quality must be given equal weightage. Instrumentation facilitates the co-ordination of various water parameters, which are essential for the optimisation of water supply and treatment plants. One of the important parameters among them is water level measurement, which is carried out at various locations viz., water reservoir, inlet chamber, open channel, alum feeding tank, lime tank, filter beds, air vessel, sump well, etc.

Maintenance of pressure instruments is essential for their proper working and accurate reading. It also improves the life and reliability of the instruments.

14.16.2 Radar Level Transmitters

They provide a non-contact type of level measurement in case of liquids in a metal tank. They make use of EM, i.e. electromagnetic waves usually in the microwave X-band range which is near about 10 GHz. Hence, they can be also known as microwave level measurement devices. However, there are some differences between radar and microwave types. They are:

(i) power levels in the case of radar systems are about 0.01 mW/cm^2 whereas, in the case of microwave systems, these levels range from 0.1 to 5 mW/cm^2;
(ii) microwaves can work at higher energy levels; hence, they are competent enough to endure extra coating as compared to radar level detectors.

A radar level detector (Table 14.13) includes:

(i) a transmitter with an in-built solid-state oscillator;
(ii) a radar antenna;
(iii) a receiver along with a signal processor and an operator interface.

The operation of all radar level detectors involves sending microwave beams emitted by a sensor to the surface of the liquid in a tank. The electromagnetic waves after hitting the fluid's surface return back to the sensor which is mounted at the top of the tank or vessel. The time taken by the signal to return back, i.e. time of flight (TOF) is then determined to measure the level of fluid in the tank.

Table 14.13: Radar Level Transmitter

Radar – Microwave type level transmitter	Service: Raw Water (Non-Contact Type)
Transmitter	
Type	Microwave level measurement
Principle	Pulse Time of Flight
Output	4–20 mA HART Current
Housing	Die-Cast Aluminium
Electromagnetic compatibility	Interference Immunity to EN 61326, Annex A (Industrial) and NAMUR
Ingress protection	IP65/IP 66/IP 67
Accuracy	±3 mm
Area classification	Non-Hazardous
Display	4-line LCD Display. Menu guided operation. Display of Envelope Curve.
Configuration	Using the keypad on display
Sensor	
Range	Liquids 0 to 5 m and 0 to 10 m depending on tank size
Temperature range	-40 °C ... +80 °C
Max pressure	3 bar abs
Materials	Sensor: PVDF Seal: EPDM
Antenna seal	FKM Viton
Process connection	Threaded or universal flange dependent on model selection
Degree of protection	IP65

In water works, various treatment processes are carried out to supply potable water. The parameters of the water which are normally used for monitoring are as follows:

(i) turbidity;
(ii) pH;
(iii) residual chlorine.

These parameters are monitored either by means of online instruments or by analytical laboratory instruments or both. Their relative advantages and disadvantages are as follows:

14.16.3 Turbidity Meter

Table 14.14: Advantages and Disadvantages of Turbidity Meter

Online	Laboratory Type
Advantages	
i. Turbidity continuously monitored ii. Can be hooked up for automation iii. Can be set for giving an alarm if minimum and maximum limits of turbidity are exceeded. iv. Human error in sampling is eliminated	i. Low cost ii. Simple to use iii. Portable iv. Easy maintenance
Disadvantages	
i. High cost ii. High maintenance is required	i. Does not monitor continuously ii. Human error may encounter

Online	Laboratory Type
iii. Periodical calibration is required iv. It is not portable	iii. Low accuracy
Maintenance	
i. Clean chamber and lens with fresh water ii. Microprocessor-based instrument has a self-calibration facility which is useful for periodical calibration iii. Clean sources of light	i. Clean sampling tube with fresh water ii. Bulb, standard sample tubes, and lens should be cleaned with soft cotton iii. Calibrate before carrying out the measurement iv. Calibrate with standard samples of 100 NTU, 10 NTU, and 1 NTU or calibrate with Formazin standard solution

14.16.3.1 Typical Specification for Online Measurement of Turbidity

Online sensor (Table 14.15) based on Nephelometric 90 degree scattering light method as per ISO 7027. The sensor must be an easy plug and play with digital communication based on inductive energy transfer with IP68 rating suitable for measuring range 0 to 9999 FNU/NTU with an accuracy of <2% of measured value or 0.1 FNU/NTU. The sensor shall store on-board all the calibration data and other diagnostic information. The sensor must have provision to store at least five user calibration points. The offered transmitter shall be four-wire digital with the possibility of connecting multiple sensor inputs for additional parameters – referred to as multichannel/multi-parameter type. The transmitter shall be suitable for outdoor installation with IP66/67 rating.

Table 14.15: Online Turbidity Meter

Turbidity Measurement	
Transmitter	
Type	Turbidity and suspended solids transmitter
Principle	Nephelometric measuring principle 90° NIR scattered light according to ISO EN 27027
Output	4–20 mA HART current
Supply voltage	100/115/230 V AC +10/–15%, 48 ... 62 Hz; 24 V AC/DC +20/–15%
Material	Field Housing: ABS PC Non-corrosive type
Display	LC display, two lines, with status indicators
Electromagnetic compatibility	interference emission and interference immunity acc. to EN 61326-1:1998
Protection class of field housing	IP 65
Ambient temperature	– 20 to 60 °C
Self-Diagnostic feature	Required
Sensor	
Measurement range	0 – 9999 FNU/0 – 3000 ppm/0 – 3.0 g/L
Material	Sensor shaft: PVC/PPS GF40 Optical window: sapphire Cable: TPEO
Max Process temperature	50 °C
Max Process pressure	10 bars
Temperature sensor	Integrated NTC temperature sensor
Connection	Fixed cable connection
Ingress protection	IP68

Turbidity Measurement	
Additional Certifications	Calibration certification
Resolution	0.001 FNU, 0.01 ppm, 0.1 g/l, 0.1%
Measurement error	<2% of meas. value (min. 0.02 FNU)

14.16.4 pH Meter

Table 14.16: Advantages and Disadvantages of pH Meter

Online	Laboratory Type
Advantages	
i. Continuously monitored ii. Can be hooked up for automation iii. Can be set for giving alarm for specified limits iv. Human error in sampling is eliminated	i. Low cost ii. Simple to use iii. Portable iv. Easy maintenance
Disadvantages	
i. High cost ii. Periodical calibration is required iii. High maintenance cost (replacement of electrodes) iv. It is not portable	i. Does not monitor continuously ii. Human error may encounter iii. Low accuracy
Maintenance	
i. Clean electrode with soap water or clean with 5% concentrated H_2SO_4 and 6% concentrated H_2O_2 ii. Calibrate periodically with a standard solution of pH 4 and pH 7 iii. Replace electrodes if dried up	i. Clean sampling electrode with distilled water ii. Calibrate the instrument with three standards samples, i.e. pH 4, pH 7, and pH 9.2 iii. Prepare standard samples from readily available capsules iv. Calibration may last from 4 days to 7 days

14.16.3.2 Typical Specification for Online Measurement of pH

Online pH (Table 14.17) sensor is based on potentiometric measurement using a glass sensor. The sensor shall have digital communication based on inductive energy transfer. The sensor shall have in-built memory to store calibration data and other additional diagnostic information. The offered transmitter shall be four-wire digital with the possibility of connecting multiple sensor inputs for additional parameters – referred to as multichannel/multi-parameter type. The transmitter shall be suitable for outdoor installation with IP66/67 rating.

Table 14.17: Online pH Meter

Transmitter	
Type	Glass electrode
Principle	Glass electrode with dirt repellent PTFE diaphragm
Output	4–20 mA HART current
Supply voltage	100/115/230 V AC +10/–15%, 48...62 Hz; 24 V AC/DC +20 /–15%
Material	Field Housing: ABS Polycarbonate non-corrosive
Display	LC display, two lines, with status indicators

Electromagnetic compatibility	Interference emission and interference immunity acc. to EN 61326: 1997/A1: 1998
Protection class of field housing	IP 68
Ambient temperature	–20 ... +60 °C
Diagnostic feature	Required
Sensor	
Measurement range	pH 0–14
Material	Glass
Max Process temperature	130°C
Max Process pressure	6bar
Temperature sensor	NTC/ Pt100
Connection	Inductive digital connection with transmitter
Ingres protection	IP68
Additional Certifications	FM, ATEX, CSA
Resolution	pH 0.01, Temp 0.1 °C
Measurement Error	±0.5% of Measuring range

14.16.4 Residual Chlorine Meter

Table 14.18: Advantages and Disadvantages of Chlorine Meter

Online	Laboratory Type (Lovibond Type)
Advantages	
i. Continuously monitored ii. Can be hooked up for automation iii. Can be set for giving alarm for specified limits iv. Human error in sampling is eliminated	1. Low cost 2. Simple to use 3. Portable 4. Easy maintenance
Disadvantages	
i. High cost ii. Periodical calibration is required iii. High maintenance cost (replacement of membrane) iv. It is not portable v. It requires electricity of battery or solar	i. Does not monitor continuously ii. Human error in sampling may encounter iii. Low accuracy
Maintenance	
i. Clean membrane if it gets clogged ii. If the membrane is damaged replace it with the new one iii. Fill up/ DPD membrane /electrolyte if necessary iv. Calibrate it using Potentiometric electrode	i. Clean tubes with distilled water ii. Calibration is not required as it is a comparator

14.16.4.1 Typical Specification for Online Measurement of Chlorine

Online chlorine analyser (Table 14.19) is based on Amperometric (membrane-based measurement of active chlorine converted to free chlorine by means of pH compensation)/DPD Colorimetric method for Free Chlorine monitoring. The sensor shall have digital communication based on inductive energy/RS485 transfer which will withstand moisture, corrosion, and ensures reliable data transmission. Complete measuring system includes chlorine sensor, pH sensor/in-built pH compensation/pH buffer with suitable flow assembly for mounting these sensors along with the transmitter. The offered transmitter must be IP65, Interference emission and immunity as per EN

61326-1:2006.

Table 14.19: Online Chlorine Meter

Transmitter	
Type	Free chlorine
Principle	Amperometric/DPD Colorimetric measurement of free chlorine
Output	4–20 mA/MODUBUS/PROFIBUS HART
Supply voltage	100/115/230 V AC +10/–15%, 48 ... 62 Hz; 24 V AC/DC +20/ –15%
Material	Field housing: ABS PC Fr
Display	LC display, two lines with status indicators
Electromagnetic compatibility	Interference emission and interference immunity acc. to ISO EN 61326: 1997/A1: 1998
Protection class of field housing	IP 65
Ambient temperature	-20….+60 °C
Sensor	
Measurement range	0.01 – 5ppm free chlorine
Material	Sensor Shaft: PVC
	Membrane: PTFE
	Membrane cap: PBT (GF30); PVDF
Process temperature	+2ºC…..+45 °C
Max process pressure	1 bar
Temperature sensor	NTC/ Pt100 for Amperometric method
Connection	Inductive digital connection with transmitter
Ingress protection	IP 68
Resolution	0.01 mg/L
Measurement error	±5% of measuring range

14.16.5 Total Dissolved Solids/Electrical Conductivity

The amount and composition of TDS in water is linked to the electrical conductivity of the water. As such, of the most effective ways of measuring TDS in water is by measuring the conductivity of the water itself. Electrical, or specific, conductivity of water is directly related to the concentration of dissolved ionised solids in the water. Ions from the dissolved solids in water create the ability for that water to conduct an electric current, which can be measured using a conventional conductivity meter or TDS meter.

14.16.5.1 Typical specification for online measurement of TDS/EC

Conductivity measurements are used routinely in several applications as a fast, inexpensive, and reliable way of measuring ionic content in a solution. The measurement of product conductivity is a typical way to monitor and continuously trend the performance of water purification systems. Ions from the dissolved solids create the ability for water to conduct an electrical current, which is measured by the online TDS analyser, and immediately displayed as sodium chloride ppm or mg/L or µS/cm conductivity.

Online conductivity/TDS analyser having the specification to measure analyse and control conductivity, TDS, and temperature:

(i) three programmable on/off relays: two for conductivity and one for temperature;
(ii) accepts 2-wire or 4-wire conductivity cells for K=0.1, K=1 and K=10 cell constants for improved flexibility and accuracy;
(iii) displays three selectable conductivity and TDS ranges per cell constant for a total of 18 ranges;
(iv) automatic temperature compensated (ATC) conductivity readings.
(v) programmable temperature coefficient;
(vi) simultaneous display of conductivity, temperature, cell constant, temperature coefficient, relay status and transmitter current mA output;
(vii) convenient one-point calibration and calibration data are stored in memory and is ready for use on power up;
(viii) standard ¼ DIN size;
(ix) performs self-diagnostic at power up to ensure proper operation.

14.17 Leakage reduction and continuity of supply

There are multiple causes for loss of water in transmission pipelines which include leakage, metering errors, public usage such as firefighting, and theft. The most critical route for losses is a leak, as they are considered to contribute an estimated of 70% of water loss in water transmission systems, this value is expected to become higher in undermanaged networks.

The two classes of leak detection system can be defined as follows:

(i) Static leak detection systems: are systems that rely on sensors and data collectors that are placed within the water network and on valves and are capable of transmitting periodical data to the network management office. This data can be used to identify, localise, and pinpoint leaks.

(ii) Dynamic leak detection systems: are systems that rely on moving leak detection devices to suspected leakage area to perform an investigation. Therefore, they rely initially on suspicion of an existing leak. Another approach is performing regular surveys around cities to identify leaks as soon as possible. Those systems can confirm the existence of leaks and immediately localise and pinpoint them.

The main distinction between the two classes is that static leak detection systems can inform the water network management of the existence of a leak almost immediately, whereas dynamic leak detection systems are required to have information of a leak possibility so that they can be mobilised for investigation. On the other hand, dynamic leak detection systems can pinpoint the exact location of a leak almost immediately under ideal operating conditions, whereas static leak detection systems will provide a location within a certain area, and they are also more prone to false alarms. The two classes encompass a wide variety of technologies to provide an accurate leak detection system, but the technologies are not limited to one class. For example, acoustic technologies, specifically noise loggers, can be dynamic and be moved from one location to the other periodically, or they can be left in the network to detect leaks.

Identification of leakage is performed by a PIC microcontroller to automate the process. Leakage and suction of water are identified by the pattern of water flow rate which is monitored continuously by the processor and if the same goes beyond a predefined threshold level, suitable alarms or annunciators are triggered to notify the leakage or theft. The flow rate is sensed by the signal conditioning unit when the water is passed through the pipeline. The sensor operates under certain pretend value. When there is a sudden drop in the water flow due to leakage or any pumping of water through motor, it will be detected by the water flow sensor. The signal conditioning unit is used to give the desired input signal of the ADC. The analogue signals generated due to variation in the flow of water sensed by the water flow sensor are converted into digital signals using analogue to digital convertor (ADC) and this digital signal is given to the microcontroller. This microcontroller enables the transmitter signal for intimation to water supply personnel. At the same time, they enable the driver unit to close the solenoid valve.

14.18 Telemetry and IoT Systems

IoT and telemetry work in tandem to provide a combination of data acquisition, analysis, storage, and reporting. Additionally, IoT systems allow operators to monitor and control system command functions from a remote location. It's a powerful blend of hardware and software technologies that improve efficiency, cost, and productivity. IoT is a web-based platform for real-time monitoring and analytics of geographically distributed systems. Telemetry is a technology which allows the remote measurement and also, reporting of information of interest to the system designer.

14.18.1 Geographical Information System (GIS)

The Geographic Information System (GIS) is a software system with which location-based mapping and characterisation of the geographic features can be integrated into a Decision Support System (DSS) for planning, execution, and management of any spatial both the manmade and natural features. GIS will play an important role in mapping, monitoring, and operation of water supply schemes under AMRUT on IoT based sensors. The areas in which GIS can be used in the water supply system from planning till operation and management monitoring are mentioned below:

(i) mapping of surface and ground water sources and Infrastructure including mapping of IoT sensors;
(ii) water resources availability estimation and determining carrying capacity through GIS;
(iii) water budgeting and audit at different levels of hierarchy;
(iv) planning of Water supply infrastructure up to home;
(v) management of system (resources, assets, functionary);
(vi) Use in day-to-day effective maintenance management by locating the areas for trouble shooting in pipe network and infrastructure through GIS-based simulated hydraulic models;
(vii) real-time monitoring of services using IoT based sensors and analytical tools with GIS linkage for water quantity, quality, operations, and maintenance. Dissemination of information to beneficiaries/public and system management group;
(viii) GIS-based Consumer Relations Management (CRM) using geospatial database of consumers and using several electronic media like mobile, e-mails, social media, etc.;
(ix) monitoring of the CRM and grievance redressals in relation to KRAs of concerned official;
(x) nation, state, ULB level visualisation of status of services through GIS enabled dashboards in the map form;
(xi) DSS development on daily and seasons basis for policy makers. In the present context of IoT sensors-based monitoring in AMRUT.

GIS for mapping and geocoding of water supply schemes, its sources, geographical boundaries of beneficiary areas, IoT based sensors deployed for the scheme for monitoring, areas under influence of a sensor. This shall, after analytics, display visually on the GIS map-based dashboards about the quantity and quality of water supply in an area under the influence of geocoded sensor.

The data from geocoded ground water level measurement sensors and its GIS-based analysis will have to be projected on the maps as dashboard for use of water scarcity management measures and DSS for the different administrative units.

Development of modules for use of the geospatial IoT sensor-based available data for effective operations management of individual water supply system by mapping the entire transmission and distribution network and using GIS-based simulated hydraulic models with real-time flow and pressure data inputs through sensors.

14.18.2 Telemetry

Telemetry is the automatic recording and transmission of data from remote or inaccessible sources to an IT system in a different location for monitoring and analysis. Telemetry data may be relayed (Figure 14.11) using radio, infrared, ultrasonic, GSM, satellite, or cable, depending on the application.

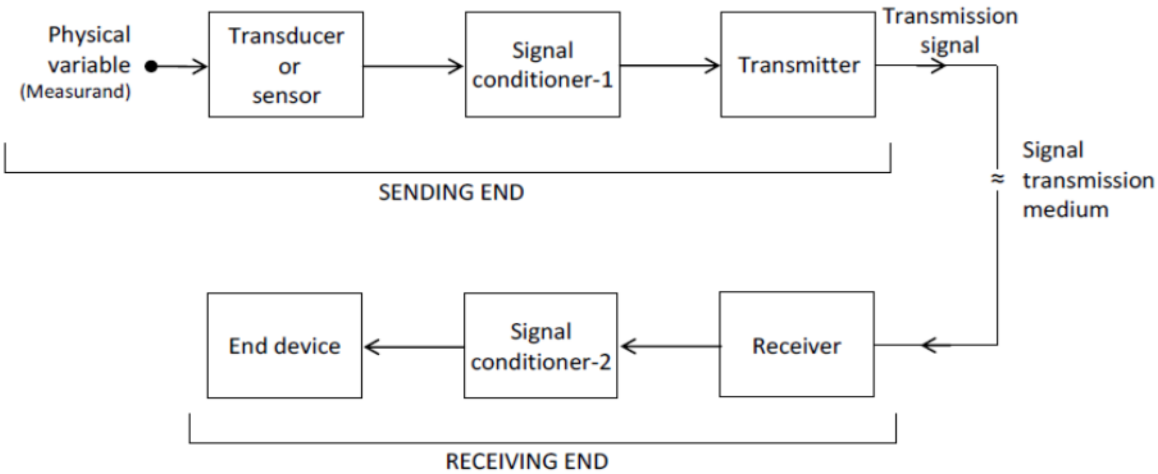


Figure 14.11: Block schematic of basic telemetry system

A high-level solution has to be described as shown in Figure 14.12.

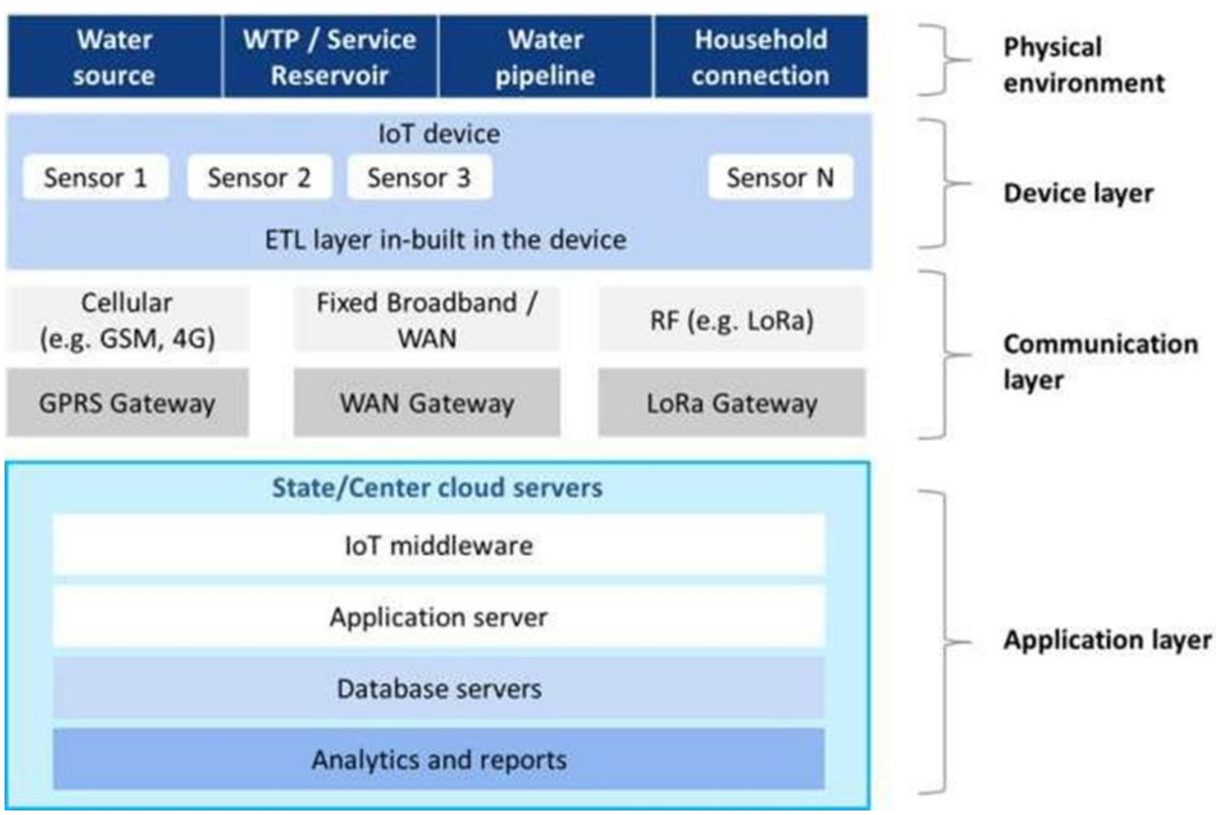


Figure 14.12: Block schematic of basic telemetry system

Networking and communication standards

The solution design should adhere to open standards and be technology neutral. It can use a variety of networking technologies as per local conditions and availability of services, such as fixed broadband/Wi-Fi, local area RF (e.g. Lo Ra), and/or cellular technologies (2G/3G/4G/5G and NB-IoT). The communications to cloud can be done via several IoT-compatible and specialised protocols.

Data for collection by Telemetry

Telemetry works through sensors at the remote source which measures physical (such as pressure or temperature) or electrical (such as current or voltage) data. This is converted to electrical voltages that are combined with timing data. They form a data stream that is transmitted over a wireless medium, wired or a combination of both.

Processing data from Telemetry

At the remote receiver, the stream is disaggregated, and the original data displayed or processed based on the user's specifications.

14.18.3 Cloud-Based IoT System

As elaborated in earlier sections, IoT systems are enterprise systems that utilise the latest technologies like internet, cloud, AI/ML, among others. In these systems, the system's infrastructural cost is reduced by implementing IoT through cloud computing. Maintaining, as well as integrating these systems, is easy compared with others.

In real time, the condition of these systems can be reported through cloud computing. Therefore, the implementation of algorithms like intricate control can be done that are frequently used on usual PLCs. Here are some key features of cloud-based IoT systems explained in Figure 14.13.

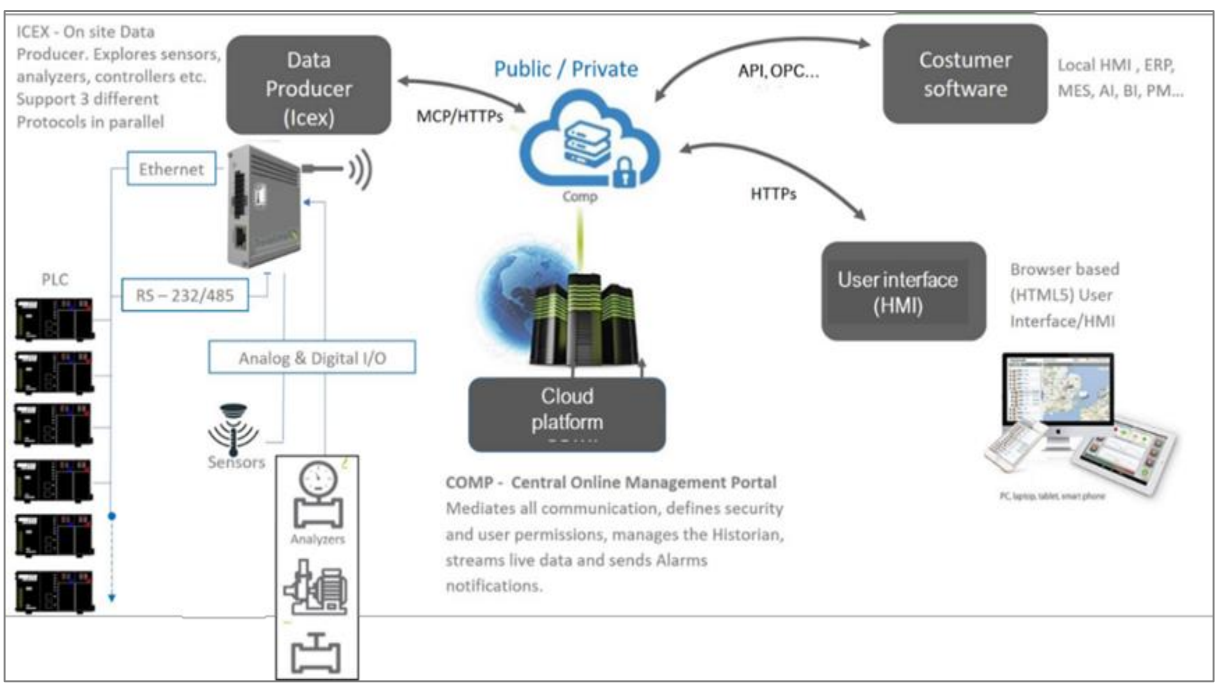


Figure 14.13: Cloud-based IoT systems

1. Cloud-based architecture: A cloud-based IoT system uses cloud computing technologies to store and process data, rather than relying on local servers. This allows for scalability and flexibility,

as well as cost savings since companies don't need to invest in expensive hardware and maintenance.

2. Communication protocols: Cloud-based IoT systems use a variety of communication protocols to connect to sensors, controllers, and other devices. These may include OPC UA, MQTT, and others such as MCP propriety protocol.
3. Security: With the increasing use of cloud technologies in industrial settings, security is a top priority. Cloud-based IoT systems use a variety of security measures to protect data and systems from cyber threats, such as firewalls, encryption, and access control.
4. Analytics: Cloud-based IoT systems can leverage the power of cloud-based analytics to provide real-time insights into operation processes. This can include predictive maintenance, anomaly detection, energy management and other machine learning algorithms.
5. Mobility: Cloud-based IoT systems can be accessed securely from anywhere, using any device with an internet connection. This allows operators and managers to monitor and control processes and assets from anywhere in the world.

Cloud-based IoT comparing to legacy SCADA.

There are several reasons why cloud-based IoT systems are considered better, more secure, and more open for the future than legacy SCADA solutions.

1. Security: Cloud-based IoT systems have better security measures in place compared to legacy SCADA solutions. The cloud service providers invest heavily in securing their systems, implementing strict access controls, encryption, and monitoring to prevent unauthorised access and data breaches.
2. Scalability: Cloud-based IoT systems are highly scalable and can easily accommodate changes in the number of devices or users. They can also handle a larger volume of data and processing requirements, making them suitable for larger and more complex systems.
3. Accessibility: Cloud-based IoT systems provide remote access to the data and control systems, allowing operators to monitor and control the system from anywhere in the world with an internet connection.
4. Interoperability: Cloud-based IoT systems are built with open standards and protocols that make them more interoperable with other systems and technologies. This makes it easier to integrate them with other systems, such as IoT devices, AI, and machine learning algorithms.
5. Cost effectiveness: Cloud-based IoT systems are more cost-effective than legacy SCADA solutions as they require less hardware and software investment. Also, maintenance and support costs are significantly lower as the cloud service provider takes care of most of the maintenance and updates.

Interfacing IoT with GIS

IoT system can interface with a GIS to provide location-based data and visualisation. Here are some examples of how IoT can interface with GIS:

Asset management: to provide a map-based view of assets, such as equipment and infrastructure. This can enable more efficient asset management, such as tracking maintenance activities, optimising routes, and identifying areas for improvement.

Real-time monitoring: to display real-time monitoring data on a map. This can provide a visual representation of the data, making it easier to identify trends, patterns, and anomalies.

Incident management: to provide a map-based view of incidents, such as alarms, faults, and other events. This can enable faster response times and improve incident management.

Environmental monitoring: to monitor environmental factors, such as air quality, water quality, and weather patterns. This can enable more proactive environmental management, such as identifying potential hazards and mitigating risks. By interfacing with GIS, IoT system can provide a powerful tool for managing and optimising processes. This can enable more efficient decision making, reduce costs, and improve overall system performance. Details of facility, control type, etc. are shown in Table 14.20.

Table 14.20: Details of facility, control type, etc.

Facility	Control Type	Control Parameters	Control Mode	Source
Pump / Pump Station	Pressure Control	Discharge Pressure	Local / Remote	Automation Program
	Level Control	Discharge Reservoir/Tower Level	Local / Remote	Automation Program
	Time Control	Day Time	Local	Automation Program
	Power Consumption Optimization	Demand,Pumps Characteristics,Network Topology, Power Tariffs	Remote	Optimization Algorithm
Puimp - Variable Speed Drive	Speed Control	Diacharge Flow/Pressure	Local	VFD setup
Controlled Valve	Status Control	Status	Local / Remote	Automation Program
Pressure Reducing Valve (DMA,PMA)	Fixed Pressure	Discharge Pressure	Local	Controller
	2 points Pressure	Discharge Pressure , Time	Local	PLC / Valve Pilot
	Dynamic Pressure	Discharge Pressure , Time/Flow	Local / Remote	PLC / Valve Pilot
Flow Control Valve	Fixed Flow	Discharge Flow	Local	Controller
Storage : Resrvooir/Tank valve	Level Control	Water Level	Local	PLC / Controller

14.19 Smart Water Management

Smart water management is the use of data and technology to optimise water usage, reduce waste, and improve overall efficiency. SCADA and IoT systems including digital twin can play a critical role in smart water management by providing real-time monitoring and control of water systems. Here are some examples of how a cloud-based SCADA and IoT systems including digital twin can be used for smart water management:

1. Real-time monitoring: It can monitor water systems in real time, including water quality, flow rates, pressure, and other key metrics. This can provide valuable insights into system performance and enable early detection of issues before they become significant problems.
2. Leak detection and NRW: It can detect leaks and non-revenue water (NRW) issues in water systems by monitoring and comparing pressure changes and flow rates. This can enable quick identification and repair of leaks, reducing water waste and minimising damage to infrastructure.
3. Energy management: It can monitor energy usage in water systems, optimising energy consumption and reducing energy costs.
4. Remote control: It can provide remote control of water systems, enabling operators to adjust system settings, turn pumps on or off, and monitor performance from anywhere with a secured internet connection.

For efficient operation and maintenance with no downtime of the services, an IoT-based real-time data collection, analysis, reporting, command and control system with online sensors installed across the city network has been implemented.

This includes real-time surveillance to ensure quality water supply at every home, real-time data analysis and decision making for uninterrupted water supply, data capture for preventive maintenance of water supply assets, reduction of NRW through leakage detection and control, efficient incidence management and quick resolution of issues, effective complaint redressal for enhanced customer satisfaction, mobile responsive real-time flow, pressure and quality monitoring of water supply, GIS-based assets and consumer mapping and real-time dashboard.

The smart water management has emerged as a transparent, cost-effective and user-friendly system with real-time water quality monitoring without human intervention, automated pump and pressure control compared to manual operation and on the spot automated testing and chemical dosing compared to manual practice, thus reducing the manpower requirement to a large extent. This also has lessened the issue identification and resolution time from days to few hours, resulting in high-level customer satisfaction. Further, since the system can now be operated remotely, it has become disaster resilient and during natural calamities such as cyclones, the services will no longer be disrupted. The DMA should have digital inlet pressure with pressure transmitter, inlet electromagnetic flowmeter, altitude pilot control valve with necessary ESR level transmitter, including mechanical gauge. The delivery of the ESR should be divided into required numbers of header depending upon hydraulic gradient of the sub-DMA and consumption. The sub-DMA should have electromagnetic flowmeter and dynamic control valve. Along with pressure transmitter at required point as per desired. The above instruments should be connected to DMA level PLC panel through data cable and the back-up power supply should have a DG of required capacity including emergency chlorine dosing VFD driven chlorine dosing pumps.

The distribution network of each sub-DMA should have online chlorine analyser and connected to DMA level PLC by optical dark fibre. The lane, bye-lane, street and commercial/official should have individual flowmeter along with digital online pressure transmitter which should be connected to main PLC by optical fibre through a street side PLC panel. Each DMA PLC should be connected through the dark fibre optic in star topology. Typical Command Centre and DMA Monitoring of WATCO, Puri are shown in Figure 14.14 & 14.15.

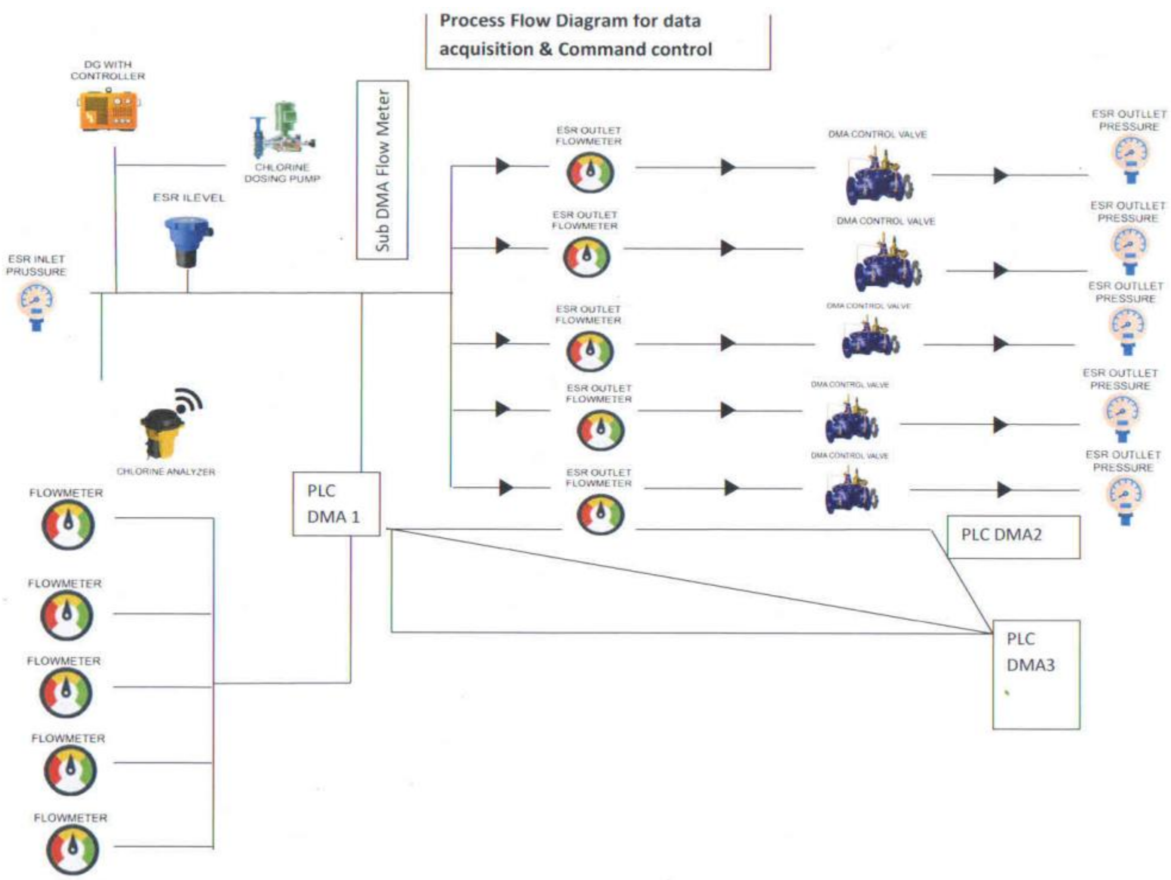


Figure 14.14: Typical Command Centre of WATCO, Puri

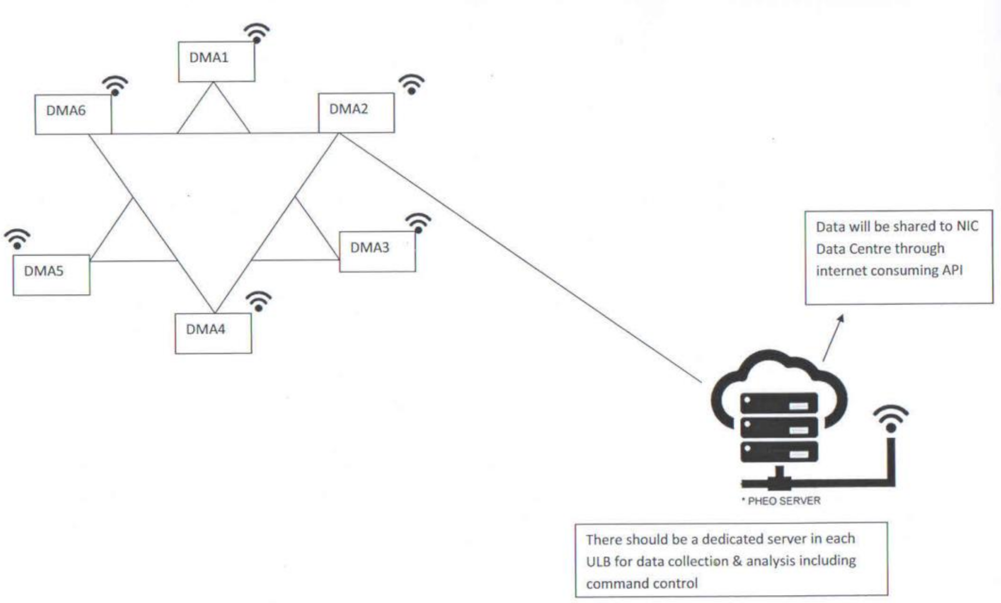


Figure 14.15: Typical DMA Monitoring followed by WATCO Puri

All DMA data should be captured in the server each at interval of 20 milliseconds per instrument round the clock. The graph and alert are generated accordingly for its monitoring and in case of emergency all functional instruments can be controlled from the command control room.

14.20 Instrumentation Matrix for Water Supply

The components, field instrumentation, electrical inputs, control system and communication technologies at various locations of the water supply system are explained in detail sin the matrix below:

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
1	**Intake pumping station (RWPS)**	Pressure Transmitter at Pump individual at Inlet Level Transmitter at Intake well Flow Transmitter at Pump outlet command discharge Temperature scanner at pumps. Vibration sensor at X Y Axis of pump Reverse rotation switch for VT pump, Pump Monitoring system Controlling and monitoring of Integral type Motorised actuator valves/gates. It is advised to select smart actuator to achieve digitalised plant. Plant communication system: PA, EPABX and CCTV Automated lighting system (Solar/EB power)	a) VFD/soft starter from LT and HT panels connected to PLC via ethernet/RS 485 Modbus/TCP IP for Energy monitoring and demand-based operation b) All multi-frequency modulation connected via Modbus communication for energy monitoring. c) Energy management d) Control and monitoring of incomer, bus coupler, individual drives, DG set e) Transformer monitoring f) Intelligent motor control centres-based panels to acquire more electrical insights	a) Automation through PLC/SCADA system b) DNP3 and HART over IP protocols are mandatory for wireless communication/field instrumentation for health diagnostics and remote parameterisation c) To achieve digital plant protocols like MQTT, MODBUS shall also be used only for specific areas d) Hot Standby CPU, redundant power supply, redundant communication-based PLC system shall be used. e) UPS with suitable battery and other sets.	Option 01 – VSAT-based wireless communication Option 02 – Multi-protocol label switching based fibre optic cable communication via service provider
2	**Raw water Rising main (RWRM) (if Applicable)**	Battery operated Electromagnetic flowmeter at tapping point Transient pressure Acoustic Transmitter (TPAT)/Online Noise sensor for continuous monitoring	-	Data logger with in-built data storage and data transfer via dual GPRS communication	Software-driven WAN-based GPRS communication with alternate service provider for redundant communication
3	**Water Treatment Plant (WTP)**	**Pre-settlement treatment** – pH analyser, Turbidity analyser and Residual Chlorine Analyser at inlet , Full Bore electromagnetic	a) VFD/soft starter from LT and HT panels connected to PLC via ethernet /RS 485 Modbus/transmission	a) Automation through PLC/SCADA system b) DNP3 and HART over IP protocols are mandatory for	Option 01 – VSAT-based wireless communication with NMS service.

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
		flowmeter at inlet pump of WTP **Filter Bed**: a) Loss of Head Transmitter for individual filter bed level measurement and Rate of Flow Transmitter at outlet of filter bed. b) SMART Filter bed automation (Wireless) **Chemical Dosing and Sludge Treatment:** Pressure instrument at each pump, Level Transmitter at each tank and Flowmeter at common discharge pump. **Post treatment** – pH analyser, Turbidity analyser and Residual Chlorine Analyser at outlet , Full Bore electromagnetic flowmeter at outlet pump of WTP. **Plant communication system:** public address, EPABX and CCTV Automated plant lighting system (Solar/EB power) Controlling and monitoring of Integral type Motorised actuator valves/gates. it is advised to select smart actuator to achieve digitalised plant. Note: It is advised to select	control IP for Energy monitoring and demand-based operation b) All Multi-frequency modulation connected via Modbus communication for Energy monitoring. c) Energy management d) Control and monitoring of incomer, bus coupler, individual drives, DG set e) Transformer monitoring f) intelligent motor control centre based panels to acquire more electrical insights	wireless communication/ field instrumentation for health diagnostics and remote parameterisation c) To achieve digital plant protocols like MQTT message queuing telemetry, MODBUS shall also be used only for specific areas d) HOT Standby CPU, redundant power supply, redundant communication-based PLC system shall be used. e) UPS with suitable battery and other sets.	Option 02 – MPLS-based FO cable communication via service provider SD WAN Connection technology (applicable for central control room and unified operation centre). Option 03 – combination of both MPLS and VSAT communication.

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
		Wireless Instruments within WTP campus for digitally enabled plant.			
4	**Clear water Pumping station (CWPS)**	Pressure Transmitter at Pump individual at Inlet Level Transmitter at tank measurement Flow Transmitter at Pump outlet command discharge Temperature scanner at pumps. Vibration sensor at X Y Axis of pump. Pump Monitoring system Controlling and monitoring of Integral type Motorised actuator valves/gates. it is advised to select smart actuator to achieve digitalised plant. Note: It is advised to select Wireless Instruments within pumping station campus for digitally enabled plant Plant communication system: PA, EPABX and CCTV Automated lighting system (Solar/EB power)	a) VFD/soft starter from LT and HT panels connected to PLC via ethernet/RS 485 Modbus/TCP IP for Energy monitoring and demand-based operation b) All MFM connected via Modbus communication for Energy monitoring. c) Energy management d) Control and monitoring of incomer, bus coupler, individual drives, DG set e) Transformer monitoring f) IMCC based panels to acquire more electrical insights	a) Automation through PLC/SCADA system b) DNP3 and HART over IP protocols are mandatory for wireless communication/ field instrumentation for health diagnostics and remote parameterisation c) To achieve digital plant protocols like MQTT, MODBUS shall also be used only for specific areas d) HOT Standby CPU, redundant power supply, redundant communication-based PLC system shall be used. e) UPS with suitable battery and other sets.	Option 01 – VSAT-based wireless communication Option 02 – MPLS-based FO cable communication via service provider
5	**Clear water Rising Main (CWRM)**	Battery operated Electromagnetic flowmeter at tapping point Transient pressure Acoustic Transmitter (TPAT)/Online Noise sensor for continuous monitoring		Data logger with in-built data storage and data transfer via dual GPRS communication	SD WAN-based GPRS communication with alternate service provider for

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
					redundant communication
6	**Intermediate Booster Pumping Station (IBPS)**	Pressure Transmitter at Pump individual at Inlet Level Transmitter at tank measurement Flow Transmitter at Pump outlet common discharge Temperature scanner at pumps. Vibration sensor at X Y Axis of pump Pump Monitoring system Controlling and monitoring of Integral type Motorised actuator valves/gates. it is advised to select smart actuator to achieve digitalised plant. Plant communication system: PA, EPABX and CCTV Note: It is advised to select Wireless Instruments within pumping station campus for digitally enabled plant Automated lighting system (Solar/EB power)	a) VFD/soft starter from LT and HT panels connected to PLC via ethernet /RS 485 Modbus/TCP IP for Energy monitoring and demand-based operation b) All MFM connected via Modbus communication for Energy monitoring. c) Energy management d) Control and monitoring of incomer, bus coupler, individual drives, DG set e) Transformer monitoring f) IMCC based panels to acquire more electrical insights	a) Automation through PLC/SCADA system b) DNP3 and HART over IP protocols are mandatory for wireless communication/ field instrumentation for health diagnostics and remote parameterisation c) To achieve digital plant protocols like MQTT, MODBUS shall also be used only for specific areas d) HOT Standby CPU, redundant power supply, redundant communication-based PLC system shall be used. e) UPS with suitable battery and other sets.	Option 01 – VSAT-based wireless communication Option 02 – MPLS-based FO cable communication via service provider
7	**Master Balancing Reservoir (MBR)**	Pressure Transmitter at Pump individual at tank Level Transmitter at top of all reservoir tank Flow Transmitter at Pump inlet and outlet of tank	-	RTU with DNP3, HART over IP and MQTT protocols	a) Data logger with in-built data storage and data transfer via dual GPRS communication b) SD WAN-based

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
		Water quality Analyser at outlet of tank Instrumentation for online dosing chlorine system (Level sensor, solenoid valve and stroke pump adjustment) Controlling and monitoring of Integral type Motorised actuator valves/gates. it is advised to select smart actuator to achieve digitalised plant. Plant communication system: CCTV system Note: It is advised to select Wireless Instruments within MBR campus for digitally enabled plant			GPRS communication with alternate service provider for redundant communication
8	**Clear water Pumping Main (CWPM)**	Battery operated Electromagnetic flowmeter at tapping point Transient pressure Acoustic Transmitter (TPAT)/Online Noise sensor for continuous monitoring	-	Data logger with in-built data storage and data transfer via dual GPRS communication	SD WAN-based GPRS communication with alternate service provider for redundant communication
9	**Elevated Service Reservoir (ESR)**	Pressure Transmitter at Pump individual at tank Level Transmitter at top of all reservoir tank Flow Transmitter at Pump inlet and outlet of tank	-	RTU with DNP3, HART over IP and MQTT protocols (Rugged RTU for remote location considering temperature and other factors)	a) Data logger with in-built data storage and data transfer via dual GPRS communication b) SD WAN-based GPRS

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
		Water quality Analyser at outlet of tank Instrumentation for online dosing chlorine system (Level sensor, solenoid valve and stroke pump adjustment) Controlling and monitoring of Integral type Motorised actuator valves/gates. it is advised to select smart actuator to achieve digitalised plant. Plant communication system: CCTV system			communication with alternate service provider for redundant communication
10	**ULB Transfer Chamber – UTC**	Pressure Transmitter at Inlet of each village Flow Transmitter at inlet of each village with data logger	-	Battery operated Data logger	a) Data logger with in-built data storage and data transfer via dual GPRS communication b) SD WAN-based GPRS communication with alternate service provider for redundant communication
11	**District Metered Area – DMA**	Pressure Transmitter at Inlet of each DMA Flow Transmitter at inlet of each DMA with data logger	-	Battery operated Data logger	a) Data logger with in-built data storage and data transfer via dual GPRS communication

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
					b) SD WAN-based GPRS communication with alternate service provider for redundant communication
12	**Sub-DMAs**	Pressure Transmitter at Inlet of each sub-DMA Pressure Transmitter at critical pressure points at low, medium and High pressure Flow Transmitter at inlet with data logger at critical pressure points Battery operated digitalised Online chlorine measurement sensor at zone-wise (Data can be directly transfer to central control room and respective OHSR without any transmitter in sensor as an alternative technology)	-	Battery operated Data logger	a) Data logger with in-built data storage and data transfer via dual GPRS communication b) SD WAN-based GPRS communication with alternate service provider for redundant communication
13	**House service connection/ commercial connection**	Option 01 – AMR Water meter at inlet of individual house service connection Option 02 – AMI Water meter at inlet of individual house service connection Option 03 – NB-IoT/LORA/RF others Water meter at inlet of individual house service	-	-	Option 01 – AMR Water meter at inlet of individual house service connection Option 02 – AMI Water meter at inlet of individual house service connection Option 03 – NB-

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
		connection Option 04 – Direct GPRS (E SIM enabled/plastic sim) Water meter at inlet of individual house service connection Option 05 – Dual communication Water meter at inlet of individual house service connection **1. GPRS** (E SIM enabled/plastic sim) **2.** During GPRS technical issue – Data can be accessed via through walkway drive method also (using suitable radio communication) Note: Suitable protocol can be used (MQTT, DNP3, CSV file format).			IoT/LORA/RF others Water meter at inlet of individual house service connection Option 04 – Direct GPRS (E SIM enabled/plastic sim) Water meter at inlet of individual house service connection Option 05 – Dual communication Water meter at inlet of individual house service connection **1. GPRS** (E SIM enabled/plastic sim) **2.** During GPRS technical issue – Data can be accessed via through walkway drive method also (using suitable radio communication)
14	**Surge Control system**	Required field instrumentation shall be provided for monitoring and controlling.	-	Via WTP PLC/RTU	SD WAN-based GPRS communication with alternate service provider for

INSTRUMENTATION MATRIX FOR WATER SUPPLY PROJECT (AI BASED DIGITAL PLANT)					
S. No	**Components**	**Field Instrumentation**	**Electrical Inputs**	**Control System**	**Communication**
					redundant communication
15	**Command Control Centre (UOC)**	Both control and monitoring of complete plant from Intake to HSC		a) Higher end FEP server, blade server, application server, Historian server, Disaster recovery management, OT/IT server, web server, HART server, LVS and redundant machines. b) Required furniture's with air condition facility c) DG and UPS supply d) Hybrid cloud management	Option 01 – VSAT-based wireless communication Option 02 – MPLS-based FO cable communication via service provider

14.21 Use of Information Technology (IT) and IT-Enabled Services (ITES)

Information technology (IT) and IT-Enabled Services (ITES) have the potential to help improve water supplies and address the systemic problems faced by the water sector. IT has great potential to provide timely information on the level of services delivered and the performance of service providers. Although snapshot baseline maps of water supplies can be created with mobile phones or GPS units, these maps often do not track core issues over time. Applied appropriately to monitoring, IT can do more to trace these issues over time and allow previous data points to be updated easily. A second use of IT is providing data management tools in areas and improving the quality of monitoring information so that it is taken seriously. Accuracy requires reviewing and validating information during data collection and reducing data entry errors. With IT, data can be checked by an expert in a utility's office even as surveys are being conducted in the field. Data do not need to be transcribed if they are entered into a phone during data collection. Validated data improve the ability of stakeholders to learn and adapt to new challenges. Additionally, data collected by different agencies can be shared for cross-validation and co-ordination. If the data are known to be accurate, broken infrastructure can be fixed in a timely manner, sector regulations can be enforced, and external support can be provided when necessary. Finally, the implementation of IT can lower the costs of existing monitoring activities by speeding up data collection, management, and analysis while reducing travel distances and costs. The cost of internet access, phones, computers, and software is decreasing dramatically while the benefits continuously improve.

ITES, is defined as outsourcing of processes that can be enabled with information technology and covers diverse areas like revenue claims processing, legal databases, content development, payrolls, logistics management, GIS, web services, etc. Information technology that enables the process by improving the quality of service is IT-enabled services. The most important aspect is the value addition of IT-enabled service. The value addition could be in the form of customer relationship management, improved database, speedy complaint redressal, etc. This radically reduces costs and improves service standards.

14.22 Application of IoT and Artificial Intelligence (AI)

The IoT is a dynamic wireless network infrastructure that integrates various communication technologies and solutions to enable the interaction between people and things/objects. This remarkable technology opens opportunities for the development of distinct applications for the cities. For instance, in the context of water management, the use of IoT allows for monitoring and controlling water supply systems in real time.

IoT systems are fourth-generation systems. In these systems, the system's infrastructural cost is reduced by implementing IoT through cloud computing. Maintaining as well as integrating these systems is easy as compared with others. In real time, the condition of these systems can be reported through cloud computing. Therefore, the implementation of algorithms like intricate control can be done that are frequently used on usual PLCs.

Artificial intelligence (AI) comprises 'a branch of computer science dealing with the simulation of intelligent behaviour in computers'. In the context of delivering efficient water supply, AI or machine learning is mainly applied to decision-making tasks: how water utilities can maximise information and data available to make better decisions while enhancing service delivery; optimising capital investment; and reducing operating costs, including social and environmental externalities.

AI in water management might come off as a huge revelation but it can change the way we treat and manage water sources around us. AI can make the process of water management easier with data analytics, regression models, and algorithms. These cutting-edge technologies help in building efficient water systems and networks. AI can be used to build water plants and to get the status of water resources. Water managers and government bodies can use AI to build a smart water system that can build efficient infrastructure for water management and can adapt to changing conditions. These systems will be cost-effective and sustainable that can optimise all water management solutions and predict potential damages. Physical water leak detection techniques are based on combining special equipment (acoustic sensors, gas tracers, etc.) with human skills. A current trend is to incorporate AI in some of this hardware to replace humans in interpreting the data (water leak noises). With the advances in numerical modelling of the hydraulics of water distribution networks, it is now possible to detect potential leaking pipe sectors through numerical methods, as long as the hydraulic models are fed with a sufficient amount of calibrated field data such as pressure, flow, and node consumptions.

14.23 Digital Twins

Digital twin is a technology which creates virtual representation (Figure 14.16) of a water supply system. In other words, digital twin creates a virtual replica of the real-world water supply system.

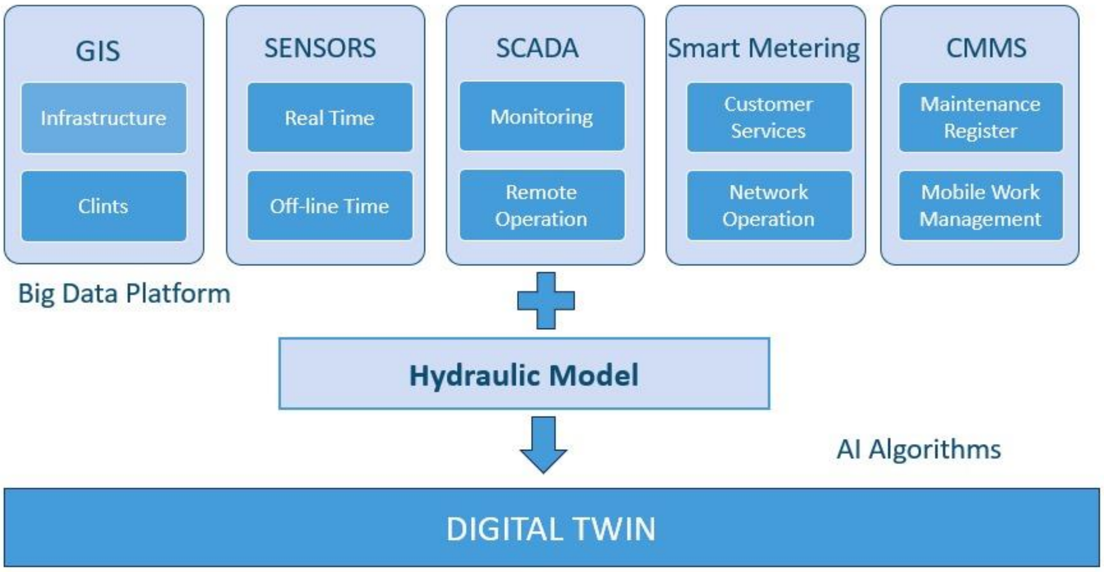


Figure 14.16: Virtual representation of typical digital Twins

Digital twin brings WTP, transmission pipeline, service reservoir, SCADA, GIS, hydraulic modelling, flowmeters, control valves and consumer meter information into a connected data environment, delivering cost-effective operations strategies in real time.

14.23.1 Objective of Digital Twin

Many times, water supply system of the towns and cities are suddenly hit by ‘unexpected’ events such as pump failure, pipe burst, low pressures, or the failure of ageing assets. On occurrence of such instances, we react only to solve the present issue. The reality is that most such events are not truly ‘unexpected’. In many cases, they are predictable, we just fail to plan and prepare. Digital twins help us, and above situation no longer remains unpredictable. With real-time data and data analytics, digital twin makes predictive analysis. For example, the flow and pressure in pipelines are continuously shown on the screen in real time with 95% confidential ratio. If we observe a sudden dip into the pressure, we can immediately know that there is a burst or heavy leakage in the system.

14.23.2 Digital Twin in addition to IoT

The purpose of the digital twin is to integrate the existing IoT solution, engineering data (e.g. hydraulic model) and IT data (GIS for example) and any other systems. The digital twin should be evergreen, live, and constantly represent the real performance of all assets and predict the behaviour of the system.

IoT systems alone provide just visibility and analytics in few points in the network. They cannot extrapolate for the whole system nor simulate the complete behaviour of the system.

The digital twin with the simulation capabilities of network software simulate the full behaviour of the systems and can ingest IoT data as initial/boundary conditions to analyse IF-ELSE scenarios, respond to emergencies (e.g. shut-off of valves, electricity failure, leakages, etc.). In addition, the digital twin technology enables detecting of events and anomalies into the network.

Digital twin can also integrate with other enterprise systems such as work order management. The digital twin solution has a capital planning module inside to help understand critical parts of the system and direct/prioritise further investment. The digital twin solution integrates analytical platforms such as PowerBI (or similar). The digital twin solution can ingest any GIS data available through Web Map Service (WMS) service to provide full context of the project and also the water networks.

14.23.3 Benefits of Going Digital

Virtual representation of water supply system of a city using digital twin spans its lifecycle. It is updated from real-time data, and uses simulation, machine learning and reasoning to help decision making. It serves as system simulation, integration, testing, monitoring, and maintenance.

The performance data of the real world of city's water supply system is taken to the virtual reality where the performance data is processed and continuously made available to the system's manager.

Digital twin provides an environment using which the utility's engineer can have access to their critical system. On observation from measurements and analytically derived results, he gets performance information of his assets. This would enhance skills of operations, maintenance, and decision making.

With real-time data and data analytics, digital twin makes predictive analysis. During O&M, if a city witnesses any problem due to pipe breakages, bursts, leakages, etc., then digital twin immediately alarms the utility and guides to take corrective measures. For example, a sudden dip is observed into real-time pressure, it indicates that there is a burst or heavy leakage in the system. Hence, the ground team can be engaged to address the issue at the site.

14.23.4 Digital Twin Setup

It is needed to configure the digital twin by assigning the data. As shown in Figure 14.17, the information is put in various silos. For example, the shape files are put in the GIS silo, while sensors and customers' information are placed in the silos allotted for them.

The hydraulic model of a city is fed to the digital twin software. For that, we have to share hydraulic model file and the GIS shape files with the support team of the digital twin. This team feeds the files to cloud platform (Figures 14.18 and 14.19) and the system automatically uploads the file and set up the model.

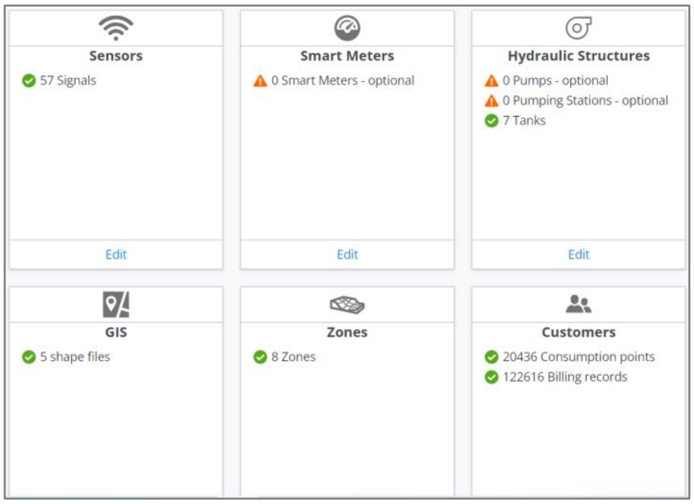


Figure 14.17: Performance data of the real world

Figure 14.18: Data uploading from city to cloud platform and response

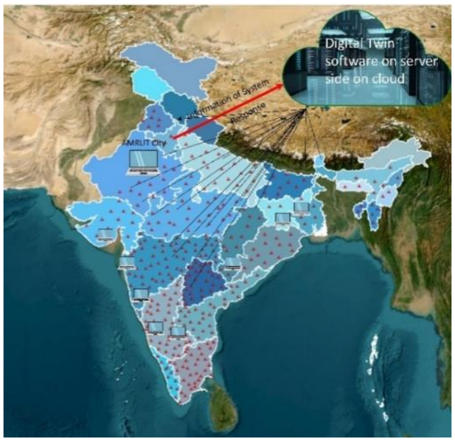


Figure 14.19: Data uploading from 500 AMRUT cities to cloud platform, processing, and response.

14.23.5 Working of Digital Twin

The working of digital twin is shown in Figure 14.20.

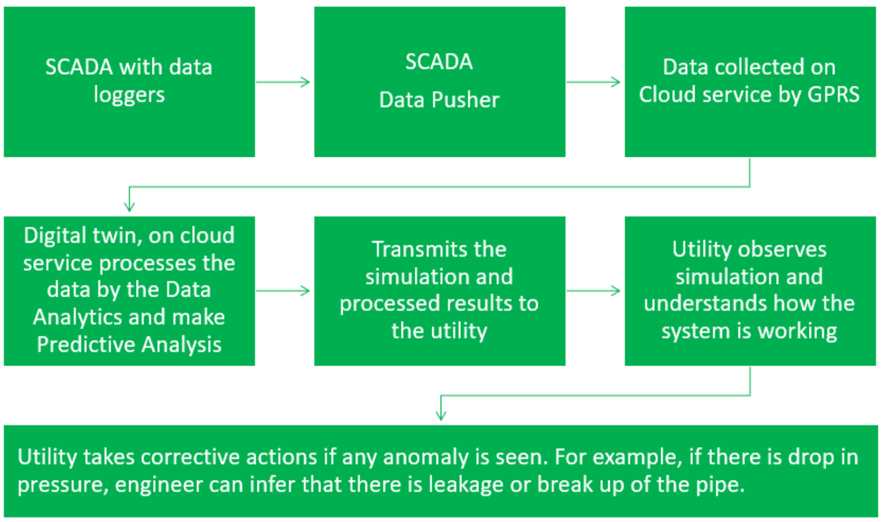


Figure 14.20: Working of digital twin.

The digital twin operation begins after the SCADA system generates data. Data pusher is inserted into the PLC of the SCADA which sends the data to cloud service. This data is accessed by the digital twin software on cloud.

Real-time simulation of the network: Once the digital twin is configurated, the digital twin shows the sensors in network, pipes, service tanks, valves, etc. It shows the real-time animation of flow through pipes as shown in Figure 14.21.

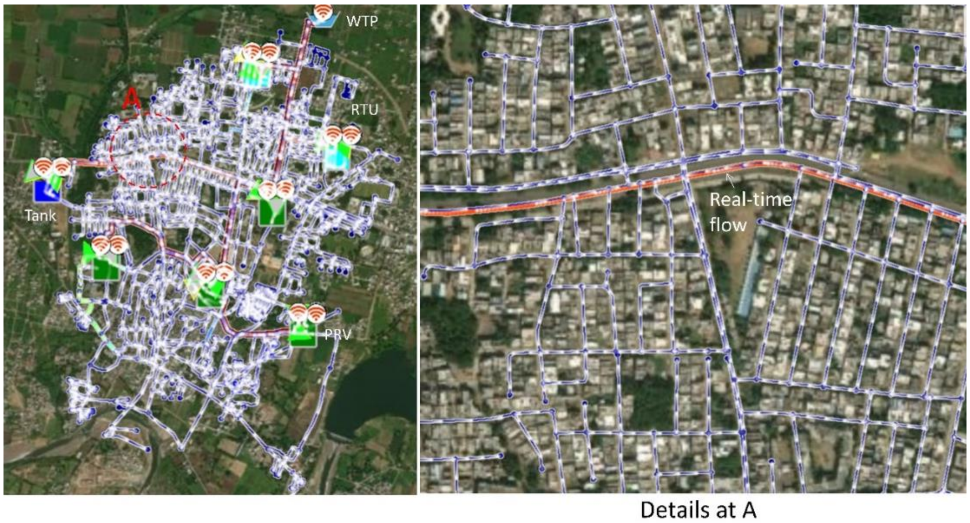


Figure 14.21: Real-time Simulation of flow in network

On configuration, the digital twin shows the machine-learned forecast and real-time results with statistical data, data analytics, and predictive analysis.

Water Audit

For real-time water audit, the customer's information is fed into the silo (Figure 14.17) earmarked for customers. The software installed on server of the cloud platform makes data analytics and computes system inflow, consumption, and losses and the NRW is computed. The results are shown in Water Balance table which is shown in Figure 14.22.

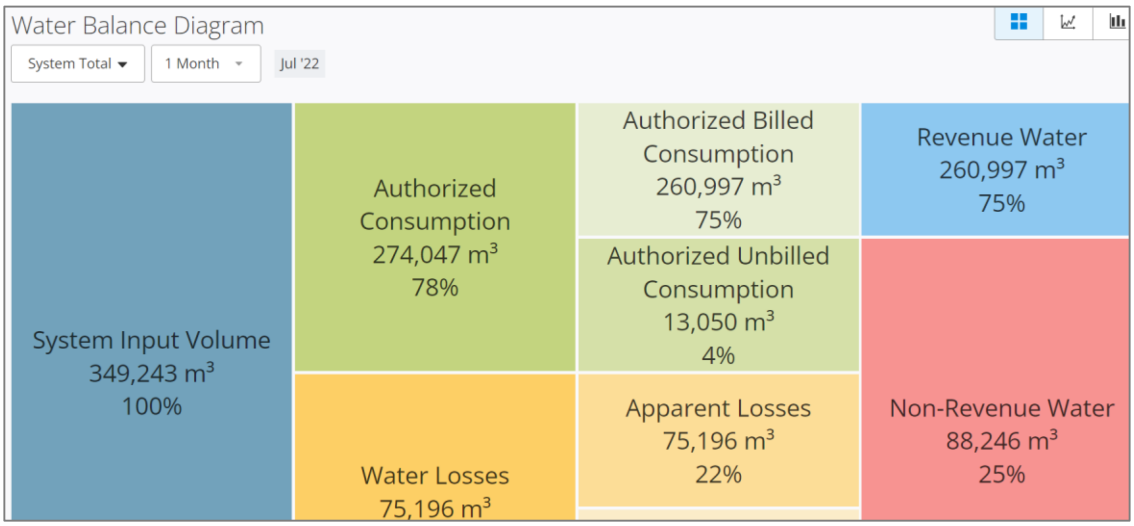


Figure 14.22: Results of water audit of the city

Digital twin produces the water balance series for different months, its graphs are shown in Figure 14.23.

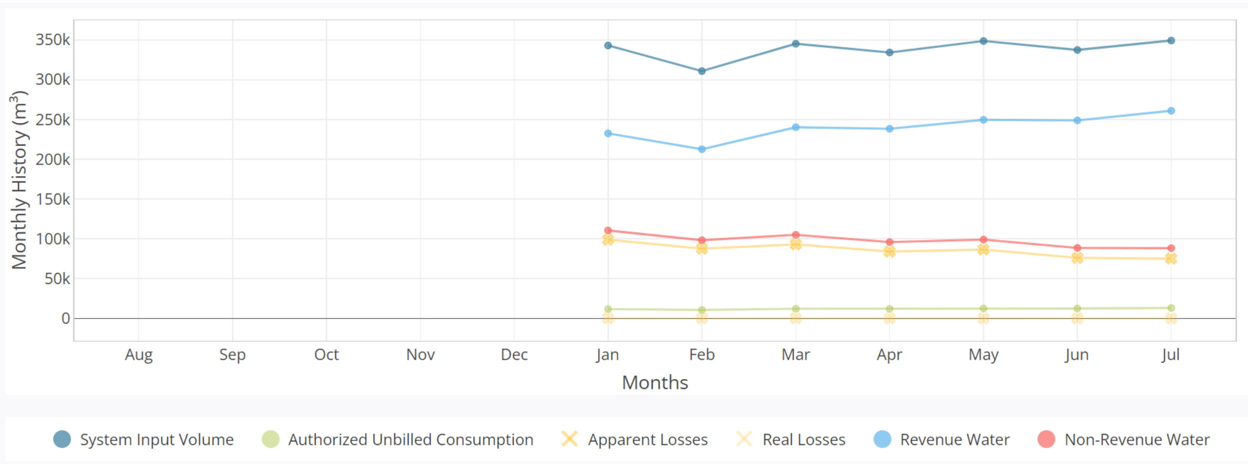


Figure 14.23: Results of water balance series for different months

Digital twin also produces water balance bar charts for various operational zones of the system as shown in Figure 14.24.

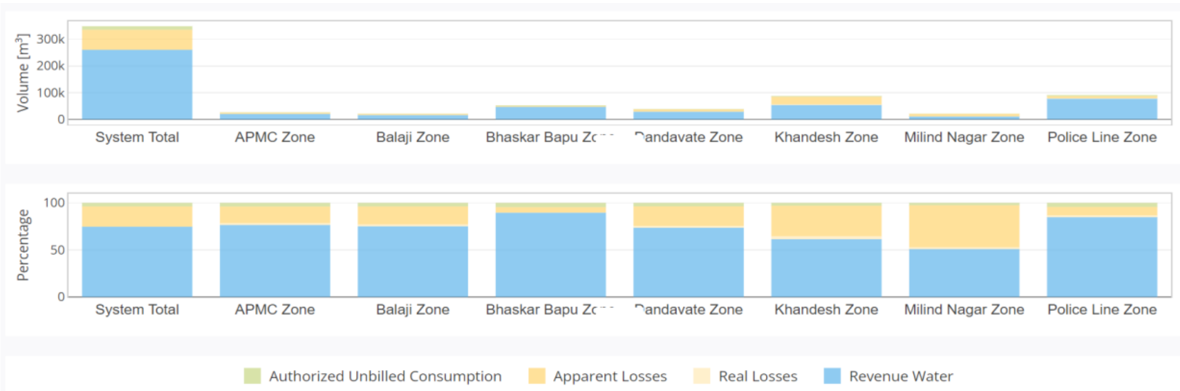


Figure 14.24: Results of water balance bar charts

Digital twin also generates minimum net night flow (MNF) values of the system as shown in Figure 14.25.

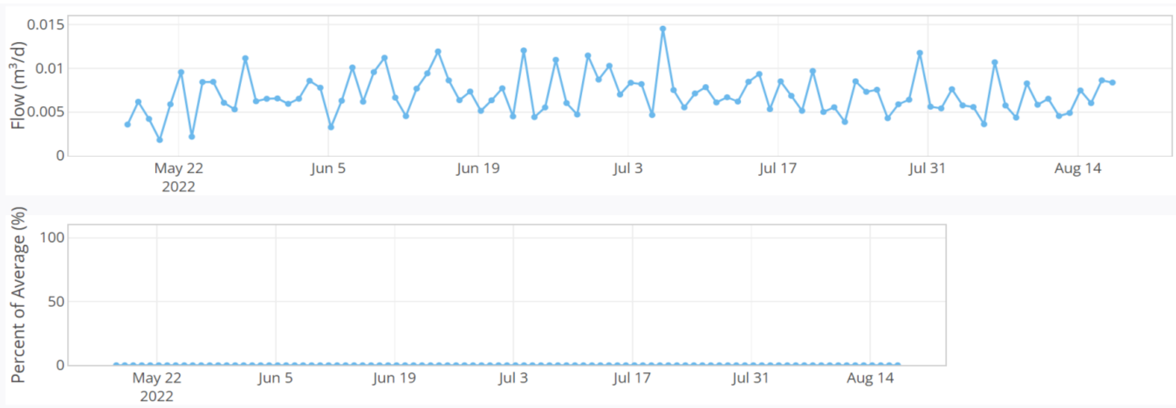


Figure 14.25: Results of minimum net night flow (MNF) of the system

Actionable Insights for the Entire Utility

It shall integrate combined data spread across multiple systems with the power of real-time analysis. The digital twin shall connect all data sources and create a continuous, consistent digital representation of your operated assets. The solution's browser-based portal shall provide an easy framework to visualise and communicate with stakeholders from any device. It shall be possible that entire team can quickly identify system inefficiencies and anomalous events, track system performance over time, make more informed decisions, and drive high-quality, consistent, and cost-effective service levels immediately as well as in the future.

Moving Beyond SCADA Results

The features allow to readily monitor various parameters at any point in the system. The digital twin visualises current data in the context of historical trends. Thematic displays provide visual cues on the normal operating ranges as well as indicate when recorded data points are outside of normal operation.

With the digital twin, it shall be possible to investigate the real-time performance for each asset using an embedded hydraulic model that is continually updated with boundary conditions from sensors. Any parameter that can be computed with the hydraulic model can therefore be simulated and monitored in real time without the need to separately open, set up, and run hydraulic modelling software. This enables graphical indication of current pressure, velocity, and other characteristics for every asset in the system, providing instant detection of areas in need of intervention to improve service levels or minimise potential issues.

Proactive Network Management

The digital twin can compute up to one week of demand forecasts for each sensor or district metered area by combining machine learning algorithms with advanced data analytics. Zone demand forecasts, along with other initial conditions from other sensors, can also be used as boundary conditions for the model runs, empowering more reliable insights and support toward a more proactive system operation.

Identification of Where Water is Going and at What Cost

The digital twin shall help to reduce NRW using live water audit calculations. It shall be possible to compare overall production against metering data to estimate how much water was lost, both in quantity and percentage. The application shall perform automated evaluation of nightly minimum flows, enabling identifying the location and quantity of NRW. This auditing shall be available for individual zones or the entire network, which allows you to detect when a problem occurred or determine the effectiveness of mitigative actions. The features shall also improve energy efficiency by leveraging real-time analyses of each pump and tank, with alerts that tell you when performance is outside of service thresholds.

Early Warning and Emergency Management

Digital twin shall improve awareness of anomalous network events such as leaks, bursts, and meter failures, contributing to reduced response times and subsequent operational cost reduction. By incorporating a real-time anomaly detection system, the digital twin shall automatically trigger alerts whenever real data is outside the expected operational behaviour. Volumes lost in each event need to be automatically computed, allowing to manage those events with status updates, category classifications, and comments. The features shall facilitate to evaluate current network performance as well as various what-if scenarios when quick decisions are needed due to a fire, pipe break, pump outage, or other time-critical events, and demonstrate the impact of actions to service levels and customers throughout the network.

Connected Data Environment

The digital twin shall leverage a connected data environment that provides a cloud-provisioned open framework for collaboration and asset information management throughout the lifecycle of water infrastructure. The connected data environment shall ensure the accuracy and availability of system data at every stage of the asset lifecycle, allowing faster project start-up, streamlined workflows, improved standard adherence, reduced risk, more informed decisions, and increased asset performance.

Water Audit

Digital twin can make following activities:

- IWA/AWWA water balance diagram – Monthly water audit by pressure zone based on production data from SCADA and customer consumption from billing data.
- NRW analysis – Automatic derivation of real and apparent losses (based on minimum nightly flow analytic).
- NRW visualisation – Graphical comparisons of the water balance components for multiple zones (historical trend series and bar charts).
- NRW key performance indicators (KPIs) – minimum nightly flow (MNF) per connection and ratio between minimum and average flow.

Pump Performance and Energy Management

- Pump operational analytics – Individual pump and/or total pump station performance evaluations in terms of best operation point, energy efficiency, and energy cost including comparisons over historical time periods (constant and variable speed pumps).
- Pump power cost KPIs – key performance indicators including total pump power cost and pump ‘inefficiency’ cost.

Tanks assessment

- Historical trend analysis – Comparative trends in tank operation (level, hydraulic grade line, and volume).
- Total Storage Assessment – Real-time storage volume assessment.
- Alerts – Low- and high-level alert notification.

Event Management and Emergency Response

- Intelligent alerts – Automatic alerts generated for sensors or zones based on user-defined rules and patterns.
- Pipe break volume KPI – automatic calculation of pipe break volume using calculated DMA flow volume exceedance above the pattern prediction.
- Events management – user updatable status, category, and edit workflow comments; ability to add manual events.
- Operational event simulation – Define and analyse impacts of events such as: valve closure/opening, pipe breaks, fires, and pump shutdowns.

Background Real-time Network Simulation

Digital twin can make following simulations:

1) 'Heartbeat' model – Re-occurring automatic background run of hydraulic model (user-defined interval such as one hour) using real-time boundary conditions, control overrides, and/or demand overrides from SCADA data derived calculations.
2) Map thematic and graphical display – 'heartbeat' modelling results for hydraulic grade line (HGL), pressure, flow, velocity, water quality, and other characteristics displayed in map view and chart views including preconfigured symbology templates and flow direction simulator on map.
3) Hindcast/Forecast simulation – Real-time model simulation of hindcast (up to 24 hours) with optional control and demand override and forecast (up to 24 hours) based on model's inherent control logic.
4) Trend chart – display SCADA vs. Model simulation results in chart.
5) Demand forecasting – Automatic calculation and adjustment of demand patterns for forecast model simulation.
6) Desktop alternative – ability to upload and download hydraulic model for offline analysis.
7) Alerts configuration – Set alerts for anomalous conditions defined based on absolute value or pattern, by individual sensor or zones, duration, and magnitude (absolute value or %); also flag lost signal or repeating values.
8) User configuration – user can incorporate new sensors, pumps, tanks, or zones into the system, configure alerts, routinely add customer billing data, and upload the most up-to-date hydraulic model as needed.
9) User settings and KPIs – Customisable settings such as production and customer water costs, pressure level of service requirement, pump efficiency, or energy tariff information.
10) User credentials – Manage users and access to cloud application.
11) Global settings – Customisable definition of thematic displays for all users.
12) External data connections – Refresh/modify links to external data.
13) Power BI integration – Integration and visualisation of user-customisable reports done in Power BI.
14) LoF and CoF factors – Define the factors that define the likelihood and consequence of failure (LoF and CoF).
15) LoF and CoF decision tree – Logic based decision tree interface to easily create simple or more complex LoF and CoF analysis; create queries across multiple datasets.
16) Risk matrix scenarios – Combine LoF and CoF using a user-defined risk matrix to create various scenarios for analysis and comparison.
17) Risk calculation – automatically performs LoF, CoF, and risk score calculation for each pipe asset.
18) Visualisation – Tabular and map display of the assets based on low, medium and high risk (or user defined); side by side comparison of different risk scenarios.
19) Capital plan – Define the performance parameters that can influence capital planning decisions; define intervention plans based on risk and performance.

14.24 Conclusion

Implementation of various solution/tools such as sensors, smart meters, advanced communication tools coupled with IoT platforms can bring about real-time database and information management and thus improving the performance of Urban Local Bodies (ULBs) and water utilities.

Smart technologies are likely to benefit both, the water utilities and the consumers, thus promoting efficient use of water both at the supply and demand side. Besides, it shall improve the credibility of the utilities thus improving revenue collection and reducing NRW. The ability to monitor water supply and distribution system on a real-time basis shall enable quick identification, prediction and prevention of potential problems such as a burst water main, a slow leak, a clogged drain or a hazardous sewage overflow/contamination. It shall thus ultimately ensure water conservation and reduction in losses and UFWs. Such systems can improve the efficacy of water supply system.

CHAPTER 15: WATER-EFFICIENT PLUMBING FIXTURES

15.1 Introduction

Water scarcity is a growing concern in many parts of the world, and it is predicted that more than half of the global population will be living in water-stressed areas by 2050. Water is a precious and limited resource, and it is crucial to conserve it to ensure that we have enough to meet the needs of a growing population. Water-efficient fixtures and fittings are an essential components of sustainable water management, and they can help individuals and communities conserve water resources. In addition to the environmental impact of water scarcity, it also has economic and social implications, as it affects food production, energy generation, and human health.

15.2 The Need for Water-Efficient Fixtures and Fittings

Water-efficient fixtures and fittings can help address water scarcity by reducing water consumption. Traditional fixtures and fittings can use large amounts of water, often unnecessarily, which can contribute to water waste.

By replacing traditional fixtures and fittings with water-efficient ones, households and businesses can save water and money on their water and electricity bills. This is particularly important in areas with high water production and supply costs, where water-efficient fixtures and fittings can provide significant savings over time.

15.3 The Use of Water-Efficient Fixtures and Fittings

Water-efficient fixtures and fittings are designed to reduce water consumption while maintaining or even improving performance. The timely monitoring of fittings, joints, and rubber washers inserted in the various fitting should be done to reduce leakages. Some common examples of water-efficient fixtures and fittings include faucet aerators, low-flow showerheads, low-flow toilets, dual-flush toilets, high-efficiency washing machines, etc.

1) **Water-Saving Faucet Aerators:** Water-saving aerators are innovative water-saving solutions/devices for washbasins, sink taps, or faucets that reduce the flow of water from faucets while maintaining the same water pressure. Figure 15.1 shows a water-saving faucet aerator.

Figure 15.1: Water-Saving Faucet Aerator

These aerators are designed with the purpose of dispensing water at a defined flow rate, say 2 to 8 litres/minute. Faucet aerators are available in various sizes and flow rates so that they can fit any faucet, and they have the following characteristics:

- save water up to 80%;
- easy to install;
- convert existing taps into water-saving taps;
- reduce water bill.

2) **Low-flow Showerheads:** These are designed to use less water while still providing a good shower experience. They work by reducing the flow of water through the showerhead while maintaining good water pressure. These showers could achieve massive water savings. Figure 15.2 shows low-flow showerheads. Regular showers flow at 15 to 20 litres of water per minute (LPM) or even more, whereas these new showers typically flow at 6 to 8 litres/minute, and they have the following characteristics:

Figure 15.2: Low-flow Showerheads

- save water up to 60%;
- equivalent bathing experience;
- easy to install;
- different showerhead spray settings;
- reduce hot water demand and save power
- available in various styles and finishes and can fit any bathroom decor.

3) Low-flow toilets: They use less water per flush than older models, which can save a significant amount of water over time. Low-flow toilets are available in various styles and designs, so they can fit any bathroom design.

Figure 15.3: Dual-Flush Water-Efficient Toilet

4) Dual Flush Water-Efficient toilets: Two arrangements/ buttons are provided for flushing, for use as per requirement, e.g., partial flush and full flush, which will save about 40% of flushing water. Figure 15.3 shows dual flush water-efficient toilet.

- This arrangement saves up to 75% of the water used for flush;
- uses as little as 4 litres per flush;
- reduce water bills.

Figure 15.4: Tank bank to save on Toilet flushing

5) Tank bank to save on Toilet flushing: Toilet flush tank banks are simple bags and can reduce almost 25%–30% of total flush water usage. Figure 15.4 shows a tank bank to save on toilet flushing. By placing the tank bank (filled with water) inside the flush tank, we displace an amount of water equal to the water in the Tank Bank for every flush. Tank bank is low-cost and an effective way to reduce water consumption in toilets.

- Saves about 2 litres of water on every flush
- No compromise on the performance of each flush

Figure 15.5: Flow Restrictors

6) Flow Restrictors: These devices restrict and limit the amount of water that is let out of an existing shower. Flow restrictors are also recommended for taps, which do not have the option to install an aerator. Figure 15.5 shows flow restrictors.

- Reduce water flow in taps and showers up to 60%

- Recommended for taps, showers, health faucet guns/hygiene taps
- Available in different flow rates

7) **Water-free Urinal Pots:** An average urinal could waste 80 to 100 Litres of water daily. Water waste in urinals can be prevented with the use of smart water-free urinal pots that efficiently perform the same task without using any water. Figure 15.6 shows water-free urinal pots.

Figure 15.6: Water-free Urinal Pots

- Save water up to 80%
- Recommended for offices, hospitals, banks and public toilets
- Save money on maintenance time and cost

8) **High-efficiency washing machines**: They use less water per load than older models, which can save up to 50% of the water used by traditional washing machines. High-efficiency washing machines also use less energy, which can also reduce energy bills. Front loading machines not only save water but also are better means of cleaning clothes in comparison to top loading machines.

9) **Water Efficiency of Cooling Towers**: Cooling water recycling or closed loop cooling systems (cooling towers) are capital intensive but can be very cost-effective solutions. Wastewater from cooling towers can be used in toilet flushing, garden watering and other industrial processes.

10) **Drip irrigation systems:** They deliver water directly to the base of plants, reducing water loss from evaporation and runoff. Drip irrigation systems can save up to 60% of the water used by traditional irrigation systems in residential gardening requirements.

11) **Use of Direct Acting Pressure Relief Valves (DAPRVs) in multi-storey buildings:** The pressures are very high in the plumbing pipelines of the multi-storey buildings causing damage/ruptures in the pipes and fittings, resulting in huge wastage of water. This can be avoided by providing DAPRVs at appropriate places in the plumbing system.

15.4 Benefits of Water-Efficient Fixtures and Fittings

Using water-efficient fixtures and fittings can provide a range of benefits, including:

- **Reduced water consumption**: Water-efficient fixtures and fittings use less water than traditional ones, which can help conserve water resources and reduce water bills.
- **Reduced energy consumption**: Using less water also means using less energy to pump and heat the water. This can help reduce energy bills and lower carbon emissions.
- **Reduced wastewater**: Using less water also means generating less wastewater, which can help reduce the load on wastewater treatment facilities and the environment.
- **Improved performance**: Water-efficient fixtures and fittings are designed to maintain or even improve performance, so they can provide the same or better experience than traditional fixtures and fittings.

15.5 BIS Standard for Water-Efficient Plumbing Products

Civil Engineering Department (CED 03) of BIS formulates Indian standards covering areas of sanitary wares and water fitting. Plumbing systems used in the country are mainly based on the water carriage system. In view of the problems of water scarcity and to reduce water wastage, these plumbing products are needed to be made water-efficient. In order to meet such a growing need, the BIS has formulated two new Indian standards, as mentioned in Table 15.1.

Table 15.1: Requirements for Water-Efficient Plumbing Products

IS 17650 (Part 1): 2021 Water-Efficient Plumbing Products Requirements Part 1 – Sanitary ware	IS 17650 (Part 2): 2021: Water-Efficient Plumbing Products Requirements Part 2 – Sanitary Fittings
Water closets/Squatting pans	**Faucets/Taps** (Lavatory faucets and sink faucets)
Flushing cisterns	
Flush valves	**Showerheads** (handheld showers, overhead showers, and handheld ablution spray)
Urinals	

The above standards cover additional requirements for assessment and water efficiency rating of sanitary wares and sanitary fittings for their water-efficient performance. These standards are to provide three types of water efficiency ratings, namely 1 star, 2 stars, and 3 stars; the higher the number of stars, the better shall be the water efficiency of the product.

Manufacturing industries are to comply with these two standards to manufacturing various plumbing products to conserve water and consumers will also become sensitised to the need for using water-efficient plumbing products.

The specific requirements for rating criteria are given in Table 15.2.

Table 15.2: Water Efficiency Rating Criteria for Sanitary Ware in India

S. No.	Product	Water Consumption per Unit	Rating Criteria		
			1 Star	2 Stars	3 Stars
Part I: Water Efficiency Rating Criteria for Sanitary Ware					
i)	Water closet/squatting pan for flushing cistern and or flush valve	a) Full flush, litres/flush	Not more than 6 L per flush	Not more than 4.8 L per flush	Not more than 4 L per flush
		b) Reduced flush litres/flush	Not more than 3 L per flush	Not more than 2.8 L per flush	Not more than 2 L per flush
ii)	Urinal	litre/flush	Not more than 3 L per flush (inclusive of pre-flush and post-flush, in case of sensor urinal)	Not more than 2 L per flush (inclusive of pre-flush and post-flush, in case of	Not more than 1 L per flush (inclusive of pre-flush and post-flush, in case of

S. No.	Product	Water Consumption per Unit	Rating Criteria		
			1 Star	2 Stars	3 Stars
				sensor urinal)	sensor urinal)
Part 2: Water Efficiency Rating Criteria for Sanitary Fitting in India					
1	Metered Faucets for Basin Use	Litres/use	1.0	0.8	0.6
	Metered Faucets for Urinal-sensor or mechanical	Litres/use	3.0	2.0	1.0
2	Wash Basin/Lavatory Faucets (also applies to sensor faucets)	Litres/Min	8.0	6.0	3.0
3	Sink Faucets	Litres/Min	8.0	6.0	4.5
4	Overhead shower	Litres/Min	10.0	8.0	6.8
5	Handheld shower	Litres/Min	8.0	6.0	4.0
6	Handheld ablution spray	Litres/Min	6.0	5.0	4.0

The House Service Connections (HSCs) are the main reason for high NRW in any water supply system. The details and the correct method for installing the HSCs can be referred in section 11.12 and 12.8 of Part A of the manual.

15.6 Bharat Tap

AMRUT 2.0 mandates all the cities to carry out reforms like water conservation through the reduction of NRW, recycling and reuse of wastewater, rooftop rainwater harvesting measures, water-efficient plumbing fixtures, energy efficiency, etc. The BIS standards, as explained above, cover requirements to be complied with by the plumbing fixtures such as sanitary ware (e.g. water closets, flushing cisterns, urinals) and sanitary fittings "(e.g. showers, mixers, taps/faucets) for their performance based on water efficiency. These plumbing fixtures were launched under the initiative called “**Bharat Tap**” by the Ministry of Housing and Urban Affairs in May 2022 (Figure 15.7).

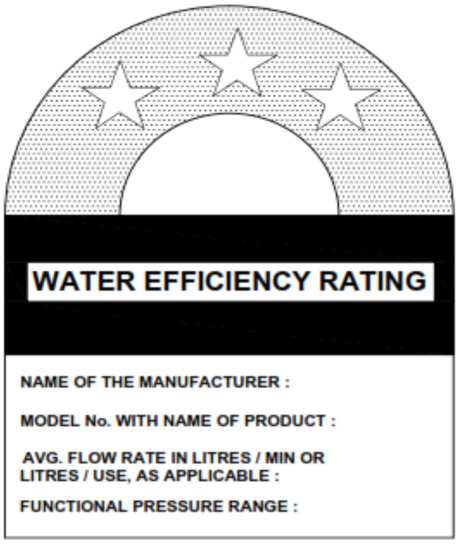
Figure 15.7: Bharat Tap Initiative

15.7 Strategies to Increase the Use of Water-Efficient Plumbing Fixtures

Major strategies that can help increase the use of water-efficient fixtures broadly include mandates, labelling and tax incentives, as described below:

(i) **Mandates**: Mandating water efficiency standards for manufacturers, new construction, replacement of old fixtures and appliances as well as mandating use of water-efficient products in all facilities.

(ii) **Labelling**: It is a certification system for water-efficient products, also known as a price tag to labels, as shown in Figure 15.8.



NOTE – The artwork of the label is only typical in nature.

Figure 15.8: Label for Water Efficiency Rating of Sanitary Fitting

(iii) **Tax Incentives**: For purchasing and installing efficient products, retrofitting, and replacing older fixtures.

The above-mentioned strategies are important, but they are only a few examples of ways to reduce residential water usage. Equally important is educating water users to make informed decisions when selecting products. In addition, ICT activities will also play a crucial role.

15.8 Conclusion

Water-efficient fixtures and fittings play a crucial role in sustainable water management by conserving water resources, reducing water bills, and providing a range of other benefits. Installing water-efficient fixtures and fittings is an easy and cost-effective way to save water and money, and it is something that everyone can do to help address the growing concern of water scarcity.

CHAPTER 16: PLANNING AND DESIGN OF REGIONAL WATER SUPPLY SYSTEMS

16.1 Introduction

Several urban water supply schemes are combinedly planned with *enrouted* villages. In addition to this, water supply schemes for peri-urban villages need to be planned. In urban water supply schemes, a common transmission main is designed and constructed to serve villages near the transmission main. However, reliability of such schemes is low. If anything goes wrong, say pipe bursts, the town or villages on downstream side are affected.

Historical observation has demonstrated that the water supply systems in India are not planned and designed with regard to the topography of the service areas or current and future water demands at nodes of the water system. Geographic Information Systems (GIS) is a tool that may assist the design engineers to account for those critical factors. It is preferable to use GIS, along with other tools, to assess transmission routing characteristics and service area demands. The Ministry of Housing and Urban Affairs (MoHUA) published an advisory on “GIS Mapping of Water Supply and Sanitation Infrastructure” in 2020, which is available at https://mohua.gov.in/publication.php?sa=manuals-and-advisories.php.

GIS based hydraulic modelling is essential tool for planning and designing of water supply schemes. It complements the complete planning and designing of the regional water supply components and network considering the topography and other physical attributes. Integration of GIS with appropriate network software should be utilised, as discussed in the Chapter 12 in Part A Manual.

This chapter also proposes to adopt 100% consumer metering in urban areas for demand management, optimise pipe diameters and equalise residual heads at ESRs/tanks of urban-rural schemes.

The objective of this chapter is to present a methodology for planning, designing, and implementing of urban-rural schemes that includes towns, peri-urban and enrouted villages.

Henceforth, combined water supply scheme for town/city, its enrouted villages and peri-urban areas shall be denoted by “Urban-Rural Water Supply Scheme”.

16.2 Problems in Urban-Rural Areas

Presently in most of the urban and rural schemes are planned with 10 to 15 villages on a single gravity main from the Master Balancing Reservoir (MBR). The result is the reliability of the water supply in downstream villages becomes less. The design of the transmission main does not ensure equal residual head at Full Supply Levels (FSLs) of Elevated Service Reservoirs (ESRs)/Ground Service Reservoirs (GSRs). Because of this, the service reservoirs closer to the input of the system experience drawl of more water than the designed demand and hence the rest of the ESRs/ GSRs do not get adequate supply. This is true especially for peri-urban villages, which are on the periphery of the urban area. This problem gets aggravated further for the remaining villages further downstream of the long transmission main. In short, long gravity transmission mains feeding many reservoirs in series causes inequality and unreliability. This conceptual shortcoming can be effectively overcome by the concept of Zonal Balancing Reservoir (ZBR), by properly grouping service reservoirs and providing ZBR for each group or a combined pumped ESR pressurised system. This is discussed further in in

Annexures 16.1 and 16.2 of this chapter. These annexures, i.e., case studies, show that without sacrificing economy, i.e., by keeping total cost (capital cost + capitalised cost of electricity along with O&M) minimum, it is possible to achieve above concept of ZBRs along with limiting the number of service reservoirs to four in each direction from ZBR and thus provide water to all service reservoirs equitably and reliably.

In addition to the problems of inequality and unreliability, other problems faced are uneconomic design of pipe diameter, improper pipe material, high non-revenue water (NRW) and inequitable distribution of flow to rural households due to improper design of distribution system of village. The villages at the end of the urban-rural water supply scheme do not receive adequate and timely daily water service. These problems can be effectively resolved by proper planning and design, with the help of hydraulic modelling and GIS.

16.3 Concept of ZBR

After deciding the scope, the project area should be divided in subareas as per topography of each subarea. For feeding tanks in the villages of each subarea, ZBRs are located at strategic higher elevation places of the established sub areas. ZBR systems have been incorporated to increase the reliability and avoid low velocities. Transmission main network is then designed to feed these ZBRs. The ZBRs should be fed either by the gravity from MBR or by combination of gravity/pumped pressure transmission main. For optimising cost of transmission main, adequate velocity in main stretches of transmission main are ensured and, thus, diameters are optimised. From ZBRs, water is further distributed for feeding village service tanks. Using ZBR makes it possible to limit the number of villages to six or so on each branch of pipelines, starting from ZBR in different direction.

16.4 Approach for Peri-urban Villages, Towns and Large Villages

Many revenue villages have boundaries which are common with adjoining town/city, the village is termed as “peri-urban village”. Behavioural habits, standard of living, and, consequently, water share requirements expressed in litres per capita per day (LPCD) of the dwellers in peri-urban villages is similar to the adjoining town/city. Therefore, it is difficult to classify dwellers as villagers. In many cases, the revenue villages have developed an attachment to the adjoining town/city. The population growth is very much influenced by rapid urbanisation of adjoining town/city. The influence is somewhat decreased if the dividing border is a river or water body and increases if the dividing border is highway or proximity of railway station. If the agricultural lands are diminishing and getting converted into plots of residential or commercial use, or if the cost of agricultural land is increasing substantially, then it indicates that population is going to increase rapidly. All these factors need to be considered and when population needs are forecasted for design. Considering these villages as future wards of adjoining town/city and forecasting its population for immediate and ultimate stage with the method of “forecasting of future ward wise population density based on equivalent area” is recommended. The forecasting process has been detailed in **Annexure 2.7** (Chapter 2) of this manual. The method mentioned is recommended to be used for assigning nodal demands and designing distribution system thereafter.

16.5 Approach for Enrouted Villages

It is a common observation, in many cases, from the trunk main going to town, there are tapping points from which enrouted villages draw water. This arrangement of giving multiple tapping points lower sustainability and hydraulic efficiency of water supply system of the town for which the project is constructed. Instead of this type of arrangement, it is recommended to provide a minimum number of ZBRs on the route of main line and feed the tanks of enroute villages from the ZBRs.

If the raw water main is designed for an urban-rural scheme, the water demand of enroute villages should also be considered while planning the transmission main through the villages.

Also, while designing an augmentation scheme of urban towns/cities, the demand of enroute villages, if they cannot be served by groundwater, surface water should be considered and integrated project should be framed. The proportionate cost for urban and rural areas shall be borne by the respective departments based on the water demand for urban and rural areas.

16.6 Holistic Planning of Urban-Rural Water Supply

For planning sustainable urban-rural scheme, it is necessary to adopt a holistic approach, i.e., from whole-to-part approach. This approach is depicted in Figure 16.1. This approach considers the overall situation of water availability in river basins, therefore, the river sub basins and watersheds in river basins should be considered.

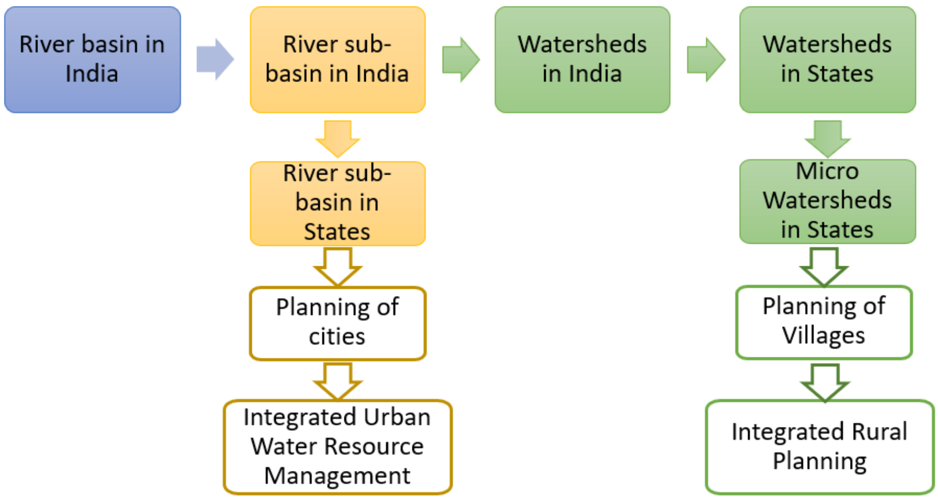


Figure 16.1: Whole-to-Part approach

16.7 Types of Urban-Rural Water Supply Schemes

There are many regions/areas in India that have acute water scarcity coupled with the 'hot spot' water quality problems like excess of total dissolved solids (TDS), hardness, chloride, fluoride, iron, nitrate, and sulphate. If we consider individual villages with such problems, the cost of standalone schemes for such villages including cost of treatment is expensive. Hence, such villages can be grouped together along with nearby urban towns and planned together to take advantage of economies of scale. Normally, in such situations, when nearby sources are not available, urban-rural is thought of with distant sources. Combined urban-rural Water Supply Schemes are categorised as follows:

a) Urban-rural with enrouted villages of cities
b) Urban-rural with peri-urban villages
c) Regional Rural Water Supply Schemes (RRWSS)

16.7.1 Design Approach for Enrouted Villages of Urban Scheme

Typical Urban-rural scheme with enrouted villages is shown in Figures 16.2 and 16.3. The components of urban-rural schemes are source, transmission main from source to WTP, transmission main from WTP to MBR, transmission main from MBR through a zonal meter to the villages, transmission main continuing from the village to ESRs beyond the villages to ESRs in the operation zone, and Operation Zone ESRs into the OZ distribution system as shown in Figure 16.2. The rest of the components for a ZBR approach are shown in Figure 16.3.

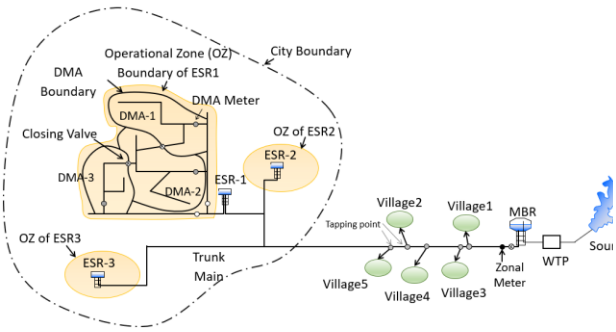


Figure 16.2: A typical urban-rural scheme with enrouted villages with tapping points (not recommended)

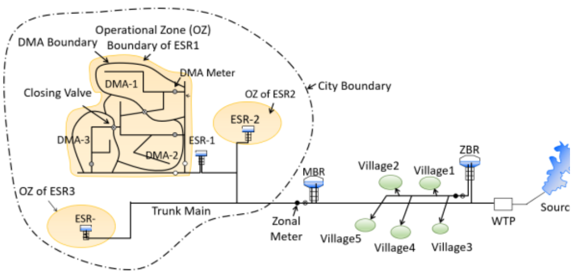


Figure 16.3: A typical urban-rural with enrouted villages with ZBR (recommended)

Figure 16.2 shows water supply arrangement to the enrouted villages through direct tapping points from the trunk main. This arrangement results in excessive residual head at FSLs of ESRs by which village ESRs can draw many times more water than they are designed for. Moreover, this arrangement of giving multiple tapping points lowers sustainability and hydraulic efficiency of water supply system of the town for which the project is constructed. Thus, water supply of the town and villages on downstream side is affected. Therefore, this type of arrangement should be discouraged, and an arrangement shown in Figure 16.3 is recommended in which the water supply to the enrouted villages is through the ZBR.

16.7.2 Design Approach for Peri-Urban Villages of Urban Scheme

Urban-rural schemes with peri-urban villages are of two types: (i) Common source along with city and (ii) separate source.

i. Common Source: A typical urban-rural with peri-urban villages scheme with common source for both urban and rural areas is shown in Figure 16.4. In this type, source of the city and the enrouted villages are common.
ii. Separate Source: Urban-rural with peri-urban villages with separate sources for urban and rural areas is shown in Figure 16.5.

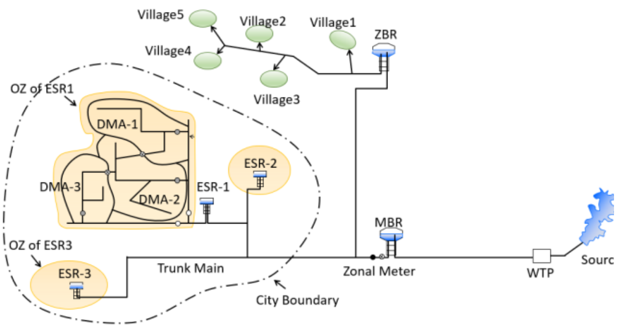


Figure 16.4: A typical urban-rural with peri-urban villages with common source

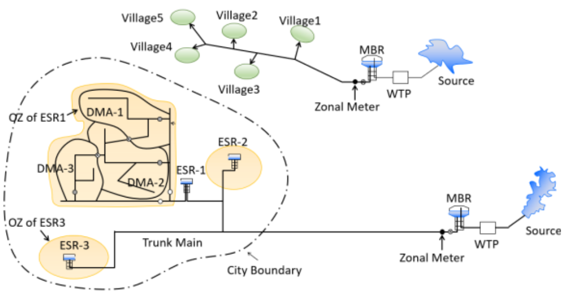


Figure 16.5: A typical urban-rural with peri-urban villages with separate sources

16.7.3 Design Approach for Regional Rural Water Supply Schemes (RRWSS)

The configuration of RRWSS is shown in Figure 16.6. In this type of configuration, water from WTP is distributed radially by locating ZBRs at central and at strategic locations of groups of villages so that every pipeline from ZBR can serve a group of minimum number of villages.

A typical RRWSS with rural villages is shown in Figure 16.6. In this type, pipelines from ZBR are designed for catering water supply of a group of minimum number of villages (say three to four villages) by placing ZBR at key strategic locations.

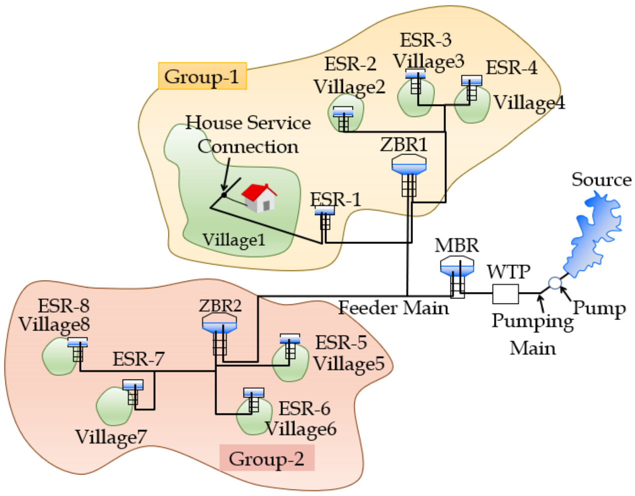


Figure 16.6: A typical RRWSS with Rural Villages

The distribution system for above categories and peri-urban villages is recommended to be designed by dividing it in hydraulically discrete operational zones (OZs). A case study of RRWSS is discussed in **Annexure 16.2**.

16.8 Design Parameters

The design parameters presented here are for all the three types of urban-rural schemes which are:

a) Urban-rural with enrouted villages of cities
b) Urban-rural with peri-urban villages
c) RRWSS

Design parameters of intake works, design period, and population forecast of urban component shall be as mentioned in Table 2.7 of the Chapter 2 in Part A of this manual.

16.8.1 Population Forecast of Village

Two cases are possible: (i) Census data not available and (ii) Census data is available.

(i) Census data not available: Population forecast of village shall be made as per Jal Jeevan Mission Guidelines.

(ii) Census data is available: The forecasted population for immediate and ultimate stage should be computed. From the census data, at least for the last three decades, forecasted population with well-known methods of arithmetic increase method, incremental increase method and geometrical increase method should be computed and generally arithmetic increase method or incremental increase method may be considered depending on the nature of the village(s). However, local body may take the decision for further review in case there are other developmental activities, as below:

a) The villages, which are on the existing highway, and which are expected to be on proposed highway, proposed railway station, proposed industries, near the industrial belts, expected or existing sugar factories or food processing units, proposed economic corridor, etc., grow at much faster rate than the computed by above-described method of averaging.
b) Due to establishment of above factor, where the growth in the last census has already occurred, the computation by the above method shows much higher forecast and needs to be realistically attenuated.

If the rate of growth is decreasing, the decreasing rate growth forecasting method, as stipulated in Chapter 2, shall be adopted.

16.8.2 Ward Wise Distribution of Forecasted Population

If the present population of the town or a large village exceeds 5000, it is first forecasted for intermediate (15 years from the base year) and ultimate (30 years from the base year) stages. Then this population should be distributed ward wise by forecasting population density for intermediate and ultimate stages. This method has been described at length in **Annexure 2.7** in Chapter 2 of this Part A manual.

16.9 Testing Pressure of Transmission mains

Field testing of pressure pipes: Field test pressure shall be a maximum of:

- 1.5 times maximum sustained operating pressure;
- 1.5 times maximum pipelines static pressure;
- maximum sustained operating pressure plus maximum surge pressure (in case of pumping mains);
- sum of the maximum pipeline static pressure and the maximum surge pressure, subject to a maximum equal to the work test pressure for any pipe fitting incorporated;
- testing pressure in accordance with the provisions of IS of relevant pipe material used.

16.10 Air Valves on Transmission Main

Air valves shall be installed at summits in the lines and at changes in grade to steeper slopes in line. Otherwise, in long horizontal stretches, it should be installed at a minimum interval of 500 metres. L-section of transmission mains shall be prepared at an interval of 50 metres. However, the interval of 30 metres shall be adopted when variations in ground levels are more. Effective diameter of air valve shall be 1/4 to 1/5 of the internal diameter of the pipeline (Ref IS: 14845-2000).

Air valve may be fixed on the shafts so that their hydraulic efficiency increases. At the location of air valves, a suitable flanged T shall be provided and, on that flange, with help of flanged pipe, the shaft of suitable height above ground shall be provided. On top of flange, the air valve shall be fixed. The proper height above ground is necessary so that miscreants cannot tamper the ball of air valve. Because of the sizable height, leakage can be observed from a considerable distance, allowing for a timely repair. Tee of air valve should be embedded in concrete block so that the shaft gets stable and is not easily disturbed by miscreants and stray animals. Another advantage of this type of arrangement is that there’s no need for a cage to protect the air valve.

16.11 Break Pressure Tank (BPT)

Design is included in section 6.14 in Part A of this manual.

16.12 Per Capita Supply at Consumer End (LPCD)

Per capita supply shall be as follows:

(a) City/town: As per Table 2.7 in Part A, Chapter 2.

(b) Rural part: As per operational guidelines for the implementation of Jal Jeevan Misson-2019, Ministry of Jal Shakti, the service level of potable drinking water supply should be at least 55 LPCD. States may enhance the same to higher level depending on availability of drinking water sources for which additional financial resources that may be required, will be met by the state government or local community or donors. In addition to this, cattle troughs may be constructed to provide drinking water to livestock especially in hilly terrain, drought prone and desert areas. It is necessary to reserve water for domestic needs from multi-purpose reservoirs/storages in consultation with concerned agencies/departments.

16.13 Capacity of MBR and ZBR

The storage capacity of MBR for Urban area shall be designed for three hours of ultimate demand & for combined Urban & Rural as well as for Rural the storage capacity shall be three hours of ultimate demand. However, ULBs are free to carry out the capacity of MBR based on the mass curve.

The storage capacity of zonal balancing reservoir in rural areas shall be designed for 2 hours capacity of the ultimate demand of the service tanks under its command area.

16.14 Losses

Breakup of losses is shown in Figure 16.7.

(i) Losses in WTP towards washing of filters, etc. = 3%

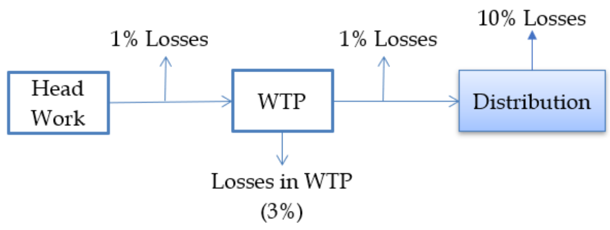


Figure 16.7: Breakup of losses

(ii) Losses in distribution = 10%

(iii) Losses in raw water main up to WTP 1% + losses in pure water up to ESRs 1%, i.e., total 2%. This is correct and adequate for normal expanse of urban-rural scheme. But when urban-rural comprises of many villages, then losses need to be considered as under.

Expanse, i.e., length from the source to farthest village via WTP is more than 20 km, then total loss should be considered at the rate of 1%, instead of 2%, per 10 km.

16.15 Hours of Pumping and express feeder of electricity

Express electricity feeder from its source of at least 11 KV substation is necessary to be provided for urban-rural scheme. It should be provided at head works, WTP, and at sumps provided for pumping to ZBRs where total operative HP is 50 or more at any of these pumping stations. Where the operative HP is less than 50, the express electricity feeder should also be provided but it can be from reliable source of electricity and need not be from 11 KV substation. Cost of these feeders should be included in the total cost of urban-rural scheme.

For rural part of the scheme, hours of pumping may be 16 to 20, depending on the availability of quality electric supply, whereas the urban and peri-urban part of the urban-rural scheme should be designed for 22 hours pumping.

16.16 Peak Factor

(a) Water distribution networks of urban-rural schemes: A peak factor of 2.5 irrespective of population should be adopted in urban areas and a peak factor of 3 irrespective of population should be adopted in rural areas.

(b) Pumping/Transmission mains: In urban-rural schemes, if power availability is 24 hours, the peak factor may be considered as 1, and if power availability is 16 to 20 hours, the peak factor may be considered as 1.5 to 1.2, depending on the specific circumstances.

16.17 Consumer meters

100% metering is necessary for urban-rural schemes to sustain the O&M of the system.

16.17.1 Water Tariff – Tool for Demand Management

For demand management and for enforcing social justice, instead of flat rates, volumetric tariff with telescopic rate structure is mandatory for towns and villages.

This is required for controlling demand and hence, it is an important tool for demand management. As every family is to be supplied water through house connection, i.e., no public stand-posts, quantum of first slab of telescopic structure should be such designed that the urban/rural poor can get drinking water at affordable price, i.e., this slab is to be subsidised accordingly.

Quantum of subsequent slab should be so designed that the middle-class persons get incentive for decreasing their consumption. At the same time, this slab should not be too costly to poor to maintain minimum hygiene standards. Quantum of subsequent slab/slabs for higher consumption shall be such priced that it becomes penalty for luxurious consumption.

16.17.2 Strategy for Solving Metering Problem

Problem in metering need to be solved instead of dispensing with meters. The following should be the strategy:

a) The cost of meters also needs to be reduced by research and by large scale production.
b) Include cost of household meters alone in the project cost. The details regarding cost recovery and replacement of meters after warranty period can be seen in the metering policy mentioned in section 13.2 in Part A of this manual.

16.18 Bulk Metering

Bulk metering at head work, inlet, and outlet of WTP, inlets of ZBRs, inlets of ESRs are essential for supply management.

16.19 Minimum Diameter of Pipe

For urban and peri-urban areas, the minimum internal diameter of primary pipes shall be 100 mm for plain area and 80 mm for hilly areas. For secondary pipes in small lanes of hilly areas for facilitating with the HSC pipes, the diameter shall be between 32-63 mm as per the local conditions. For other rural areas, the minimum diameter of pipe shall be outer diameter of 63 mm in plain areas and 50 mm for hilly areas.

16.20 Design of Raw and Treated Water Mains up to tank

16.20.1 Pumping Mains

a. Raw water: Pumping Mains are designed with well-known method of economical diameters. In this method, the logic is based on the minimisation of total cost which comprises of costs of pumping main, cost of pumping machinery and capitalised cost of pumping expenditure based on consumption of electricity. For this exercise, instead of taking subsidised cost of electricity, actual unit cost of electricity needs to be considered.
b. Clear water: Optimisation of pipes has been discussed in Part A, Chapter 6 (Transmission of Water). While carrying out the exercise, when pipe diameter decreases, velocity increases and frictional loss (m/km) also increases, which in turn, increases the HP and pumping expenditure. By iterative process, the optimised diameter is thus computed. In short, the guiding tools are velocity (m/s) and head loss gradient (m/km) in the pipeline.

The design of pumping main, which pumps water to multiple service reservoirs, has been discussed in the Part A, Chapter 6.

16.20.2 Gravity Mains

The same tool of optimisation of pipes, i.e., velocity (m/s) and head loss gradient (m/km) needs to be used in the design of diameters of gravity mains as vividly and as prudently as it is used in the design of pumping mains.

Normally, diameters, lengths, pipe material, LSL of MBR, FSL, and demands of ESRs are fed as data and the software analyses the data and computes the residual head at the inlets (FSLs) of each ESR to be served by that MBR.

All pipelines from source to ESRs shall be laid on all-season roads and branch roads to WTP, MBR and ESRs shall be provided in the project along with its cost.

16.21 Residual Nodal Head in distribution system

(a) It shall be as per Table 2.7 of Part A, Chapter 2 for the cities.
(b) For peri-urban areas, the minimum residual head shall be 15 m.
(c) Jal Jeevan Mission allowed use of stainless steel flow control valve (FCV) to control flow and pressures in house service connection. As this FCV reduces the residual pressure at ferrule, the residual pressure in village schemes that use such FCV shall be 12 m.
(d) However, for village schemes that are not using such FCVs, residual pressure shall be 7 m.

16.22 Capacity of ESR

Capacity for ESRs of urban-rural schemes shall be as per the Table 2.7 of Part A, Chapter 2.

16.23 Fire Requirement

(a) Urban part: It shall be as per the Table 2.7 of Part A, Chapter 2.
(b) Enrouted villages: It shall be same as per (a) above, however, the value of population (P) shall be considered as population (in thousands) of the ultimate stage (30 years) of the villages under jurisdiction of each ZBR

Water for extinguishing fire outbreak in villages should be stored in ZBR.

16.24 Number and Location of Isolation Valves

It shall be as per the Table 2.7 of Part A, Chapter 2. For enabling effective breakdown maintenance of leaky pipes in distribution system, isolation valve at appropriate locations should be provided in adequate number to isolate the network. As-built drawing showing the locations of isolation valves that should be readily available to maintenance staff.

16.25 Control valves

Control valves such as pressure reducing valve (PRV) and flow control valve (FCV) are vital for equitable distribution of water and equal terminal pressures.

16.26 Pipe Material

Distribution system – Provide metallic and/ or non-metallic pipes as per the site and service conditions.

Raw/treated water pumping mains, transmission mains and feeder mains, are the arteries of water supply projects and preferably be laid with metallic pipe having internal lining. If non-metallic pipes are proposed, they shall be duly justified.

Gravity transmission mains, inside and outside city areas, should be based on economical size of the gravity mains. The metallic pipes shall be preferred. If non-metallic pipes are proposed, they shall be duly justified.

16.27 Laying of Pipelines

Minimum cover of 0.9m is recommended, however cover should be provided as per respective BIS code for different pipe materials & suiting to the local field conditions. Laying of pipe shall be as per procedure for laying given in Part A, Chapter 11. In colder region (like Ladakh, J&K, etc.) the pipes have to be provided with thermal insulation.

16.28 Flow Computation

Flow can be computed using Hazen-Williams method or Darcy-Weisbach method.

16.29 Hydraulic Model for urban-rural scheme

It is described in the Part A, Chapter 12 (Service Reservoir and Distribution System).

16.30 Designing of distribution system

It is described in the Part A, Chapter 12 (Service Reservoir and Distribution System). However, operation zones and district metered areas (DMAs) are discussed as below:

16.30.1 OZ and DMAs for Urban and Peri-Urban Areas

Operational zone (OZ) should be hydraulically discrete, and each OZ is the jurisdiction of each ESR/GSR to serve water supply. Performance of distribution of water depends on size of OZ of ESR/GSR or its jurisdiction. A typical concept of OZ of urban-rural scheme is shown in Figure 16.8. The OZ is subdivided in district metering areas (DMAs).

DMA: DMA is a hydraulically discrete subdivision of a large network, created by closing of isolation valves interconnecting the surrounding network and thus isolating area. DMA receives water from bulk pipeline coming from ESR and supplies continuous water through 100% metering of consumers.

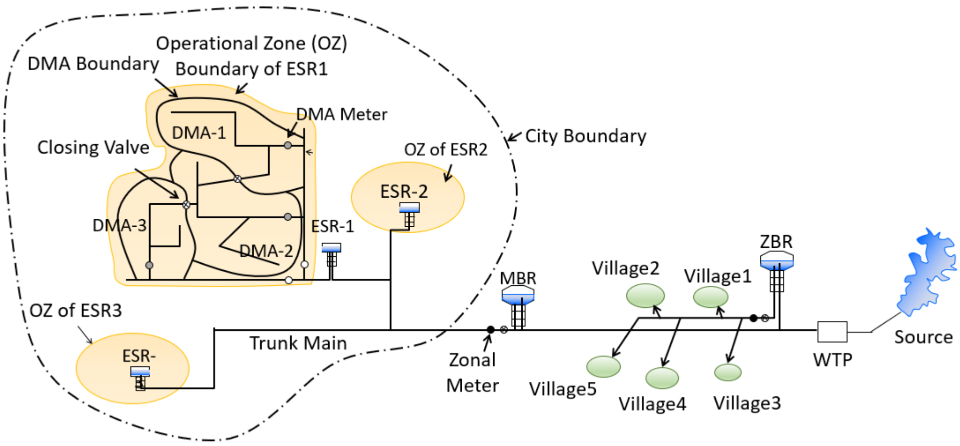


Figure 16.8: A typical urban-rural explaining concept of OZ and DMA

The design criteria for OZ and DMA, along with the detailed design procedure of distribution system, is explained in Part A, Chapter 12 (Service Reservoir and Distribution System).

16.30.2 For enrouted villages and RRWSS

Each village shall be considered as individual operation zone for which one service tank is supplying water. If the size of the village is big enough (more than 500 to 3000 connections in plain areas and 500 to 1500 connections in hilly areas) then DMA shall be considered, as given in Part A, Chapter 12.

16.31 Design OF Transmission Mains of Urban-Rural

Detailed design procedure of transmission main is discussed the Part A, Chapter 6 (Transmission of Water).

16.32 Solar Water Pumping Systems

Due to the pumping systems, energy consumption represents the biggest part of recurring energy expenses in the drinking water & sewerage sector—sometimes up to 70%, impacting the O&M sustainability. Erratic availability of power / voltage may lead to potential damage or reduced lifespan of the pumps. In terms of reaching the absolute poorest, this service delivery model should include the affordable water price & include subsidies to support most marginalized households where required. Hence an alternative solution for sustainable energy supply should be sought in the field of Renewable Energy Sources (RES). Among the renewable energy sources, solar energy is the most widely used source in water & sewerage sector, as such availability of solar resources are abundance in most part of the country. The economic benefits and performance of Solar Photovoltaic Systems depend strongly on the global irradiation of the geographic location where the system is installed and local climatic & environmental parameters also have an impact. Solar Energy Systems requires negligible maintenance & with no recurring power expenses except regular check-ups hence it is considered an appropriate alternative solution. Water Supply System has to meet water needs constantly at required level of reliability over the whole planning period. While planning water supply systems, conjunctive use of solar – powered water system will be explored to reduce the recurring energy cost of water supply & sewerage systems. Reduction of conventional energy use and CO_2 emissions in the urban water supply system must be achieved without compromising the water needs of all users.

The available options can be:

1) Stand-alone System – for small systems,
2) Hybrid System – can be operated with solar energy during the sunny hours only. Possible energy bills saving can be in between 40-50%
3) Grid Tied System with net metering basis – Solar energy so generated, to be transmitted to the grid possible energy saving can be plus 90%.

The grid tied systems can be located either on-site or distant locations subject to availability of land. For the urban & regional water supply & sewerage schemes, Hybrid / Grid tied system / both in combination can be considered as per techno-economic feasibility analysis based on the site specific conditions & requirements. Hence optimal system sizing & selection (various components i.e. Pump, Photovoltaic array, Inverter, transformers etc.) can be carried out in consideration of lean period insolation, available sunny days over the year & local environmental conditions in consideration with techno-economic viability. Possible installation options are:

Fig No. 1

Fig No. 2

Fig No. 3

Fig No. 4

Fig No. 5

Fig No. 6

Figure 16.9 (1 to 6): Possible Installation Options

Key design requirements are as follows

- Data Collection – Geographic Location, Design Period, Designed Water Demands, Lean Period Total Dynamic Head, Site Specific Sunny Days & Worst Month Peak Sunshine Hour
- Design Process –
    1) Pump Electrical Energy

       Energy in kW = $\left[\frac{Q*TDH}{102*\eta}\right] PSH$

       Where Q = Flow rate (LPS); TDH = Lean period Total Dynamic Head; q = Combined Efficiency of Pump & Motor (as per manufacturer); PSH = Peak Sunshine Hour

       Note: Pump can also be designed using available softwares
    2) Array Sizing for standalone System (AS in kWp) = EE *(1 + Losses in percentage)
    3) Array Sizing for Grid Tied System (AS in kWp) = [Annual energy requirement / (No. of sunny days X Lean month PSH)] X (1 + Losses in percentage)

Example –

A. Off Grid Stand Alone System

Given Data:

Design Population — 1000; Water Supply Level — 65 LPCD; Total Design Head — 60 m; Peak Sunshine Hour — 5 hours; Wire to water efficiency — 58%; Water Density — 1000 (kg/m^3); Specific Gravity of Water — 9.81(kga/s^2); Solar Panel Array Losses — 40%

Calculation:

Water Demand (Q) = 65 m^3/day;

Energy in kW= $\left[\frac{65*1000*60}{(3600*24)*102*0.58}\right] 5 = 3.81$

Pump Electrical Energy (EE in kW) = 3.81 (5.1 Horse Power — commercial available 5 HP) Array Sizing in kW = 5.65 kW considering radiation uncertainties

B. On Grid –Grid Tied (Net Metering Basis) System

Given Data: Mardanpur Group Water Supply Scheme Block Budhni; District: Sehore; Capacity - 25.65 MLD; Project Cost - 26040 Lacs

Operating Horse Power - 1240 HP; Pumping Hours - 22; Average Peak Sunshine Hour - 5.5 hours; Power Factor - 0.96; Sunny Days – 300; Solar Panel Array Losses – 40%

Calculation:

Array Sizing in kW = $\left[\frac{(HP * 0.746) * \text{Pumping Hours} * 365}{\text{Power Factor} * 300 * PSH}\right] * (1 + \text{Losses in percentage})$

Required Array Sizing = 6565.21 kW; Land Requirement @ 6 sqm/kW = 39390 sqm

Annual Actual power bill cost Rs. 400 Lacs; As per MNRE Benchmark Cost, amount required for solar system installation Rs. 2365 Lacs; Payback Period – 7 years; Sufficient Land is available at treatment plant site.

Note: Present cost of production of water is Rs. 15.00 per thousand liter. With the installation of solar system cost of production of water can be reduced to Rs. 5.00 per thousand liter.

C. On Grid –Grid Tied (Net Metering Basis) Hybrid System

Given Data: Bhopal – Kolar Water Supply Scheme; Capacity – 162 MLD; Horse Power - 3000 HP; Pumping Hours – 23 hours

Required Array Sizing 16107.69 kW

Note: In this case due to limited availability of land, 6000 kW system can be installed.

- Installation Requisite – Direction true south; Orientation Site's geographical Latitude; Shadow free area; sufficient land available for installation & also future expansion; No HT line passing over & above Solar Array.

Note: Land required for solar installation may be 6 sqm /kilo watt / 6000 sqm /MW subject to solar panel efficiency. For the bigger systems land requirement may be little lesser.

Figure 16.10: Solar Installation

To enhance the solar yield an appropriate technically feasible & cost effective tracking system is to be provided such as motionless stationary tracking system. Higher sizing of panels is recommended to take care of solar radiation uncertainties as per site specific needs (10-20%). Regular washing & cleaning mechanism of the PV modules is mandatory i.e. for achieving the optimal power generation as per design over full planning period. To ensure solar system performance, IoT enabled measurement & monitoring system shall be the integral part as per site specific needs in consideration with Annexure no. I of “Specification for Photovoltaic Water Pumping System” dated 22.03.2023 issued by Ministry of New & Renewable Energy, GoI. For bigger water supply systems possibility for PPP mode can be explored. The planning period for the solar photovoltaic systems can be adopted as 15 years in-line with pump design life. As per needs later on solar panel array can be further augmented. Energy yield is dependent on efficiency of the solar modules hence quality of solar modules should be ensured as per MNRE guidelines. Decision making key requirements for adoption of solar energy system in urban water supply schemes – availability of land; technical feasibility; financial viability; life cycle cost analysis etc.

The solar water pumping system shall be considered after comparing the financial feseability of the conventional energy and solar energy sources by taking into account capital cost, O&M cost, land availability and other local conditions.

ANNEXURES

ANNEXURES

[ANNEXURE NUMBERING IS CHAPTER RELEVANT]

Annexure 2.1: Case Study

1. Puri City 24×7 Water supply under DFT Mission

The water supply infrastructure in Puri was mainly augmented under JNNURM and AMRUT schemes until 2019. The water was supplied from ground water sources on intermittent basis.

Under the DFT Mission, comprehensive assessment of infrastructure gap was carried out and zone wise DPRs were prepared for conversion of intermittent water supply into continuous water supply and migration to surface water source (Bhargavi River) followed by execution of work and installation of household connections and meters.

1. Policy Interventions

Puri enabled policy interventions which are as follows:

1) Right to water
2) Execution of connection by PHEO/WATCO as public works
3) Household need not bother to obtain road cutting permission for connection
4) Easy instalment on connection charges
5) Explicit component of community participation now 100% coverage

The administration has relaxed house connection norms for the poor which are as below:

- Water connection with indemnity bond
- Waiver of connection fee
- Providing house connection with two taps at Govt. cost under AMRUT for the slums
- Covering all uncovered slums under AMRUT and now under State Plan
- Shift from hand pump, tube wells to Piped Water Supply - equity in water distribution.

2. The Project

The scheme is executed by the Water Corporation of Odisha (WATCO). The scheme is designed for a minimum average residual nodal pressure of 14 m and a residual chlorine of 0.2 mg/L. All zonal reservoirs have been integrated with WTP. Flow, pressure, and water quality is monitored on a real-time basis. The scheme is web-monitored. For this, a dedicated server is provided for monitoring water levels in 18 numbers of service tanks, pressure, flow, residual chlorine. This data analytic work is being carried out under the guidance of National Informatics Centre (NIC) Bhubaneswar. The salient features of the scheme are as follows:

1) GIS mapping and hydraulic modelling was done.
2) GIS based consumer survey was done.
3) Infrastructure for 100% coverage of water supply was implemented under AMRUT.
4) 19 DMAs were created each with about 2000 connections.
5) Flow control valves (FCV)s, pressure reducing valves (PRVs), bulk and household meters were installed for all DMAs by investing about Rs. 32 Crores (per capita cost of Rs.1350). Total investment under JNNURM and AMRUT was about Rs.178 Crores. Overall per capita cost is Rs.7000.
6) Woman self-assistance group, called “Jalsathi”, have been engaged in billing and revenue collection and also in identifying illegal connections. They could achieve 90% collection efficiency and helped in eliminating illegal connections.

7) Per capita water supply reduced to 130 LPCD. Previously, the per capita water supply was ranging from 50 to 350 LPCD with a large inequity.
8) NRW reduced from 54% to 15%.
9) 100% metering achieved. The cost of house service and meters (up to meters) was borne by WATCO. In the new connections, saddle sets with compressor fittings were provided to prevent leakages. Due to this, water leakage came down drastically.
10) All the **household sumps are being delinked** gradually as water is reaching up to 3rd floor. Water is directly reaching from pipeline to overhead tanks and taps.
11) Sumps are permitted for the residential and commercial building having more than three storeys.
12) No additional water source is required for 24×7 water supply.

3. Brief of Project

After successful running of 24×7 in pilot zones for about a year, especially supply to slum/low-income group areas, the scheme was scaled to the entire Puri city, covering a population of 2.4 lakhs and having 32,300 house connections spread in 32 wards and 64 slum areas. All these are spread over an area of 16.84 sq. km. Added to this, Puri being a very important pilgrimage centre, on an average about 50,000 pilgrims visit the city every day. This world-famous pilgrimage city has been visited by nearly two crore people annually adding to the challenge of supplying drinking water to this large population.

Key Information of Puri city is as follows:

1) Population: 2.5 Lakh (approx.)
2) Source: River Bhargavi
3) Slum population: 66,000 with 471 connections
4) No. of households: 32,017
5) WTP: 42 MLD
6) Total water demand: 38 MLD
7) No. of GSRs: 5
8) No. of ESRs: 19
9) No. of water testing laboratory: 1
10) OZs: 19
11) No. of DMAs: 19 Nos, one per OZ with 1000 to 2000 connections
12) Length of water distribution pipelines: 275 km
13) Clear water rising main: 46 km
14) Raw water rising main: 800 metre

4. Distribution System

There are 19 DMAs in Puri city. The jurisdiction of each ESR has been considered as the area of the DMA. One service tank is shown in Figure 1. Outlet arrangement of ESR is shown in Figure 2. It is to be noted that on outlet, isolation valve followed by the bulk meter and then flow control valve (FCV) have been installed.

In Puri, each tank has one OZ, which itself acts as one DMA. There are 19 tanks and 19 DMAs in the entire city. Peak factor of 2.5 has been considered in the design of distribution system.

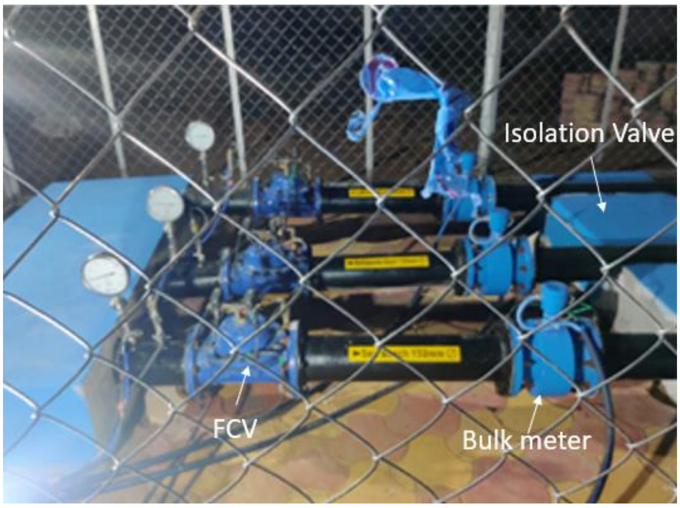


Figure 1: One service tank

Figure 2: Outlet arrangement of ESR

5. Community Lead Water Distribution

The city administration and WATCO have started in 2019 the community lead novice water supply management by partnering with self-help groups (SHGs), called as "Jalsathi." Jalsathi (Figure 3) are acting as bridge between customers and WATCO. Their partnership is based on incentives with women of SHGs.

Figure 3: Jalsathi women group at Puri

The roles and responsibilities of Jalsathi are as below:

1) facilitating new connections;
2) reading water meters, distributing water bills, and collecting user charges;
3) field water quality testing;
4) supporting consumer complaint redressal;
5) sensitising people on water conservation.

Currently, 32 Jalsathis are working on the mission in Puri, achieving 97% of revenue collection efficiency.

6. Quality Assurance through Third Party Quality Monitoring Laboratories

The State Government has established one State Level and eight Divisional Level Laboratories on PPP mode for continuous testing and monitoring of water and wastewater quality. The highlight of testing is that it is carried out by independent laboratories. Following programmes have been initiated:

Figure 4: Digital display board at public places

(i) 'Pure for Sure' Campaign: This IEC exercise has been started to create public confidence in public water supply system, so that people can drink from tap rather than relying on existing supply systems. Real-time online water quality data are displayed in public places on LCD screens (Figure 4), in order to build confidence in users.

(ii)Empanelment of plumbers: Plumbers are being empanelled (including returning migrant workers) and they are trained for installation of household connections as per pre-defined standards to avoid faulty connections and leakages.

(iii) Inclusivity approach: Water supply services to the urban poor: The State urban water policy envisions "RIGHT to WATER" for everybody. Providing water and sanitation services to the slums is one of the top priorities of the mission. It is mandated to cover the entire slum households/populations across the state under the mission in a phased manner. Convergence of the Mission with the various ongoing urban poor development programmes of Government such as JAGA Mission ensured greater reachability.

(iv) Lab on wheels: Mobile van laboratories have been deployed for on the spot quality testing for improves water quality surveillance of vital parameters and efficient incident management.

Figure 5: Quick response team

(v) NRW control cell: NRW reduction has been the major challenge. An exclusive cell has been created with dedicated crew members for leakage reduction. With focused implementation on NRW reduction, the NRW has come down to less than 15% than previous 54%.

(vi) Quick response team: Exclusive mobile crews (Figure 5) have been set up for immediate maintenance of the leakages and quick response to water supply related incidence management.

Project Cost

Project cost of schemes under various programmes is shown in Table 1.

Table 1: Project cost of schemes

Programme	Centre	State	Total	Remarks
JNNURM	72	18	90	Centre: State - 70:30
AMRUT	44	44	88	Centre: State - 70:30

Sujal (Drink from Tap)	0	46	46	State Funding (Construction Plus Metering, House Connection and Ultrafiltration WTP)
Total	116	108	224	

Duration: It took 14 months for survey, investigation, planning, design, and tendering for the DFT Mission. But execution for 24×7 WS was carried out in just nine months.

The scheme is now treated as show case for the entire country.

2. Malkapur 24×7 Water Supply System

Malkapur is a town adjacent to Karad City in Maharashtra. It has a population of 35,000, having area of 9.5 sq. km. The town has a medical college, industries and irrigated agriculture, and trade as main economic drivers.

Old system fell inadequate in capacity and coverage. People started feeling the heat, getting water once in two to three days and unscheduled. There was an increase in demand for water tankers.

A new water supply system is shown in Figure 1. Source is Koyna River which is perennial. The scheme was planned and approved for Rs. 1225 lakhs for a population of 67,196.

Figure 1: Malkapur 24×7 water supply system

The salient features of the scheme are as below:

1) Area of Operation:	Malkapur, Dist. Karad, Maharashtra
2) 24×7 scheme implementation year:	2009
3) Total cost:	14.30 crore
4) Agency:	Malkapur Nagar Panchayat (MNP) with support from the Maharashtra Jeevan Pradhikaran (MJP)
5) Pump:	150 HP
6) Capacity:	8 MLD (Million Litre per Day)
7) Coverage Area:	9 km^2

Status Before Implementation:

1) Old system was designed in 1988 for 14,000 population.
2) Old system was inadequate in capacity and coverage.
3) People started feeling the heat, getting water once in two to three days and unscheduled.

4) Increase in demand for water tank reliable, consistent, and pressurised water supply.

Pipes used in distribution network are of PE 100 grade in rolls with strictest quality assurance from raw material till finished pipe. Connections are made with electrofusion-welded PE tapping tees, ferrule, MDPE pipe of single length from ferrule to house. 100% consumer metering was done with Automatic Meter Reading (AMR) Water meter, read with drive by system. AMR data transferred to computer for billing and analysis. AMR type bulk water meter was used for online water audit. Pressure sensors are set to calibrate the hydraulic model and monitor the system health. Automation of pumps is carried out to switch them “on” and “off“ remotely using GSM communication.

The distribution system was laid by using HDPE pipes, available in 50-60 m coil (Figure 1) lengths, which reduced number of joints. The jointing was made by electrofusion method (Figures 2 and 3).

Figure 1: HDPE pipe in coil

Figure 2: Jointing of pipes by electrofusion coupler

Figure 3: Electrofusion machine

Figure 4: Pipe with tapping tee and ferrule

Figure 5: Consumer connection

Figure 6: Consumer connection by MDPE pipe

Project Innovative are as follows:

1) Taking ISO for System: Nagar Panchayat has taken ISO certification for WTP.
2) Express feeder for 24 hours electricity: For supply, 24 hours electricity through express feeder, the funds made available by the Government of Maharashtra.
3) HSC at every property: It is a slogan of Nagar Panchayat, and the rate contract is finalised for making HSC to the consumer who applied for it.
4) GIS for water supply: Nagar Panchayat has appointed an agency for making GIS for water supply including application software.

The consumer connections (Figures 4 and 5) with ferrule were done by adopting continuous MDPE pipes using electrofusion method.

Status after Implementation:

1) Cost Reduction: Automated operation of pumps, WTP and head resulted reduction in costs of operation.
2) Elimination of Bad Practice: Introduction of AMR for water billing system reduced illegal water connection, etc.
3) Service Improvement: Increase in the recovery of the costs from 60% to 80%. Water conservation measures taken place such as Telescopic tariff structure, mass awareness, discount for optimal use, etc. Reduction in consumer complaints from 200 per month to three to seven per month. Due to high and continuous pressure in water, energy bills of consumers were reduced because water reached up to fourth floor without ground level pumping.

Benefits resulting from the project:

Following are the benefits of the 24×7 water supply project:

1) improvement in delivery time and services;
2) best quality of water;
3) reduction in wastage of water;
4) simplification of billing procedure;
5) saving in electricity and helping clean development initiatives;
6) reduction in operation cost.

The scheme was awarded by the award of Hon. Prime Minister Excellency Award in the year 2011. The project has sustained 24×7 pressurised system for 14 years. ULB is taking interest and responsibility for maintaining the scheme with 9% to 11% NRW. The scheme has not only succeeded in giving benefits to the Malkapur citizens but has also provided the state and nation, a success story on 24×7 water supply for whole of the town.

3. Alnawar 24×7 Pressurised Water Supply Scheme

1. About Alnawar town

Alnavar is a fast-growing town situated in the Malanad area of Dharwad District and about 33 km from Dharwad city which is a district headquarters. Alnavar town is located at latitude and longitude of 15°25′40″N 74°44′56″E. It has an average elevation of 675 metres MSL. The town Panchayat Alnavar (TP) was constituted in 1973. The Alnavar TP has got 18 wards and equal number of councillors. The population of the town as per 2011 census is 17,288 and the present population is about 22,000 approximately.

This project of 24×7 pressurised water supply to Alnavar town from river Kaali as source is implemented by Karnataka Urban Water Supply and Drainage Board (KUWS&DB). This project comprises of infrastructure development along with reforms in water sector. The proposed project is primarily focused on providing assured bulk water supply from perennial surface water and also improving the water distribution and service delivery to the customers by providing continuous pressurised (24×7) water supply service along with DFT concept.

2. Assets created for 24×7 Water Supply System

The scheme with source as Kaali River was approved in 2017 for the cost of Rs. 71.90 Crores. The project has been implemented and 24×7 pressurised water supply started from 01.07.2022. Following are the components of the project:

1) Headworks: 8.5 m size Jackwell with 12.5 m size pump house including 3.2 m diameter intake well and 215 HP raw water pumping machineries.
2) 406 mm diameter mild steel (MS) raw water rising main from headworks near Dandeli to WTP at Javalli village with a length of 16.23 km.
3) 5.57 MLD in 20 hours WTP along with three lakh litres capacity sump cum pump house and 120 HP centrifugal pump set at Javalli village.
4) 406 mm diameter MS pure water rising main from WTP at Javalli village to ESR at Alnawar with a length of 18.88 km.
5) 150 mm diameter and 200 mm diameter ductile iron (DI) feeder mains from 406 mm diameter MS rising main to ESR in Alnavar town with a length of 0.55 km.
6) 1.00 lakh litres capacity shaft type RCC ESR on 15 m staging at Vidya Nagar in Alnawar town (two 5 lakh-litres cap existing OHTs).
7) DI (250 mm diameter) and HDPE (75 mm to 200 mm diameter) water distribution network and HSCs. Zone - 1, 2, and 3 in Alnavar town - 52.12 km.

Figure 1: Aeration

Figure 2: Clariflocculator

Figure 3: Filters

Figure 4: Online Chlorine Monitoring Point

Figure 5: Online Critical Pressure Monitoring Point

Figure 6: “DFT” Stand post with Real time Quality parameters display.

Water treatment units are shown in Figures 1, 2 and 3. Online monitoring of chlorine is shown in Figure 4. Online critical pressure monitoring point is shown in Figure 5. DFT stand post with Real Time Quality parameters display is shown in Figure 6.

Improvements in the SLBs after the implementation of the project are shown in Table 1.

Table 1: Improvements in the Service Level Benchmarks.

S. No	Indicator	Unit	Baseline Before Project	After Project
1	Coverage	%	60%	100%
2	Per Capita Supply	LPCD	70	135
3	Metering	%	Nil	100%
4	NRW	%	40%	<15%
5	Continuity of Supply	Hours/day	4 to 6 hrs. once in 6-8 days	24 hrs.
6	Quality	% water sample meeting water standards	No data	100%
7	Efficiency in Redressal of Customer Complaints	%	50%	100%
8	Cost Recovery of water supply services	%	60%	100%

S. No	Indicator	Unit	Baseline Before Project	After Project
9	Efficiency in Collection of Charges	%	70%	100%
10	Residual pressure at tap	m	In-equitable pressure	Minimum 10 m
11	Residual chlorine at tap	ppm	-	< 0.1 to 0.2 ppm

3. Key parameters adopted to achieve DFT mission

These are:

a. Real-time monitoring and control of raw water pumps at headworks, Operations of automated 5.6 MLD cap WTP, pure water pumps at WTP, inlet and outlet of reservoirs.
b. Real-time monitoring of flow from headworks, WTP and to reservoirs. Also, outflow data from reservoirs.
c. Real-time monitoring of quality parameters (turbidity, pH, chlorine) at WTP and also at three DMA's.
d. Real-time monitoring of critical pressure monitoring points of all three DMAs.
e. Real-time monitoring of all three reservoirs levels.
f. The entire system operations from source to sink real time monitoring is based on cloud/web based and android app and data transfer through GSM.
g. Android-based application "Alnavar Water Portal" was launched and dedicated on 26 March 2023 by the Hon. Union Minister of Parliamentary Affairs, Coal and Mines, GoI, to public fingertips to know the system operational status, quality of the water, and levels of the reservoirs.
h. Advanced DFT stand posts with display of water quality parameters are provided.
i. Water supply system assets are mapped with GIS including households.
j. NRW is ranging from 7% to 15%.
k. Improved collection compared to demand.
l. Customer redressal system through control room.
m. The project outcomes are analysed with due field survey/data by Centre for Multi-Disciplinary Development Research in Dharwad. The result found satisfactory.

In future, it is targeted to improve the system with implementation of the digital twin and to provide the access to respective billing and collection.

4. 24×7 W.S. of Belagavi, Kalaburagi & Hubballi-Dharwad

Karnataka Urban Water Sector Improvement Project (KUWASIP) implemented 24×7 water supply projects in the cities of Belagavi, Kalaburagi, and Hubballi-Dharwad. Government of Karnataka has approved KUWASIP programme during 2004 under World Bank assistance, with a project cost of Rs. 237.04 crores, covering 10% of the populations each of Belagavi, Kalaburagi, and Hubballi-Dharwad cities in five demonstration zones. The programme was commissioned during 2008 and successfully running since 2008. The works are taken up under Performance Based Management Contract (PBMC). The details of works taken up in these cities are as follows.

a. *Belagavi:* Laying of 94 km (south demonstration zone - 40 km and north demonstration zone - 54 km) of distribution network and provided 8405 (south - 4352 and north - 4053) metered HSCs completed.
b) *Kalaburagi:* Laying of 48 km of distribution network and provided 3280 metered HSCs completed.
a. *Hubballi-Dharwad:* (Hubballi demonstration zone - 70 km and Dharwad demonstration zone - 35 km) of distribution network and provided 13323 (Hubballi - 7541 and Dharwad - 5782) metered HSCs completed.

Continuous (24×7) pressurised water supply is being provided to five demonstration zones of Kalaburgi, Belagavi (S & N), Hubballi & Dharwad. Operation and Management w.e.f. 1.4.2010 is being carried out under a separate contract as per Karnataka Transparency in Public Procurement (KTPP) and funded by user charges of water in demonstration zones.

Details of demonstration zones under KUWASIP are shown in Table 1.

Table 1: Details of Demonstration Zones under KUWASIP

Particulars	Belagavi		Kalaburagi	Hubballi	Dharwad
	North	South			
Selected wards	44, 45 (F) 46, 48 (P)	3, 4, 5 ,6 (F) 7, 8 (P)	17,23,44,33 (F) 24,6,19,32,42 ,43, 49 (P)	27,28 (F) 29,32 (P)	8,9,10,11 (F)
Population	59173	57546	37008	65164	43795
HSCs	5745	5587	4004	10212	6951
Average per capita supply in LPCD	118	91	82.75	138.71	177.20
Average per capita Consumption in LPCD	101	83	73.14	120.45	194
Average Physical Losses since inception (%)	13.82	9.19	10.15	13.56	12.17

Situation before implementation of KUWASIP programme is as below:

1. Water supply service in the project cities was poor: insufficient water at low pressure (Figure 1)
2. Periodicity of supply: Kalburgi: One to two hours once in two days; Belagavi: Two to three hours once in three days and Hubballi-Dharwad: Two to six hours once in five days.
3. Lack of awareness on health and hygiene, water use, and management.
4. Reliance on stand posts.
5. Loss of wages, especially for women due to long queues to collect water from stand posts.
6. NRW was high (>40%)

Figure 1: Water collected from pits

Benefits achieved by successful completion of KUWASIP 24×7 water supply project are as below:

1. Continuous pressurised water supply to all the customers in the entire demonstration zone irrespective of topography.
2. Substantial reduction in leakage and hence, savings in water.
3. Availability of fresh and potable water for the entire 24 hours leading to good health and hygiene.
4. Savings in cost of pumping of water from sump as the water reaches directly to second floor.
5. No necessity of waiting for water as in case of intermittent water supply, hence, increase in per capita income.
6. Least operation and maintenance cost.
7. No necessity of storing water.
8. Decrease in generation of wastewater.
9. Attendance of students in schools has improved.
10. No public stand posts.
11. 100% customers are metered and billed.
12. All homes have become happy homes.

The KUWASIP Project has been given a National Award by Hon. President of India during 2009.

Based on the success stories of KUWASIP, there is an extension of 24×7 water supply covering the whole cities in Belagavi, Kalaburagi, and Hubballi-Dharwad under upscaling 24×7 project “Karnataka Urban Water Supply Modernisation Project” (KUWSMP) under World Bank assistance. The works are under progress.

5. 24×7 Water Supply Scheme in Ilkal Town

(a) About the Project

Under North Karnataka Urban Sector Investment Programme (NKUSIP), under ADB assistance, the 24×7 water supply scheme is completed in Ilkal town of Bagalkot district. Ilkal town is one of the towns in India having covered 24×7 water supply scheme in the whole city. The cost of the project is Rs. 27 crores. The project is completed and commissioned in 30 months. Presently, 24×7 pressurised water supply is being supplied to the whole city having 9,000 HSCs, covering a population of 60,000. 100% metering is done, and water charges are being collected from the customers for the consumed water quantity. Good response is being received from the residents of the town. 24-hour consumer complaint redressal system is introduced and is being attended on priority. Presently, the ULB is maintaining the water supply scheme in the city.

(b) Situation before and after implementation

The details are shown in Table 1.

Table 1: Situation before and after implementation

SN	Parameter	Before	After
1	Continuity	Intermittent - once in 2 to 3 days	24×7
2	Pressure	Insufficient and non-uniform	Min 7 m and uniform
3	Water quality	Not assured	Maintained as per standards
4	Billing and collection system	Flat rate	Volumetric with 100% metering
5	NRW	Not accounted (> 30%)	< 20%
6	Coverage	Partially	100%
7	Customer complaint redressal	No	Customer care support

Residents of the town are very happy with the benefits of the scheme.

6. Coimbatore 24×7 Water Supply Project

The 24×7 continuous pressurised water supply of the Coimbatore City Municipal Corporation, Tamil Nadu, is being implemented by the concessionaire: M/s Suez Projects Pvt. Limited, Gurgaon, New Delhi.

1. Project Overview:

The Coimbatore 24×7 Pressurised Water Supply Project aims to supply continuous pressurised water to Coimbatore Corporation Area (105 sq. km) for 12.35 lakhs population (2018), projected to 17.60 lakhs by 2044. Water demand: 255 MLD (2018), estimated to be 346 MLD by 2044. The scope of the project is to study, design, create, operate, and maintain water distribution infrastructure for sustainable 24×7 pressurised water supply including meter reading, billing, and collection services within city limits. The authority handles bulk water, consumer tariff, and new service connections/disconnections.

2. Key Features:

a.	Project Area:	105 sq. km (Pre-2011 Coimbatore Corporation Area)
b.	No. of HSC:	1,50,000
c.	Population:	12.35 lakhs (2018) → 17.60 lakhs (2044)
d.	Water Demand (MLD):	255 (2018) → 346 (2044)
e.	DMAs:	98
f.	Service Reservoirs	36 (Existing: 33.6 ML) + 32 (New: 68.4 ML)
g.	Distribution Pipes:	1866 km (120 km Old/Retained, 1746 km New/Replace)
h.	Project Duration:	26 years {1 year for Study + 4 years for EPC +
i.		25 years for O&M (Including 4 years of EPC)}
j.	Start Date:	08-01-18
k.	EPC Completion Date:	07-01-25
l.	Concession End Date:	03-08-44

3. Contract Model

Contract model is based on PPP with staggered annuity payment concession. The ownership and control of the water supply remains with the Authority. Concessionaire is free of risk of ‘Return on Investment’. The fund flow requirement of the authority is pre-defined. One-year study and 25 years (Including four years EPC) O&M ensures the optimal design of the infrastructure.

4. Present Status

The concessionaire is operating and maintaining the existing distribution infrastructure, including meter reading, billing, and collection for the past four years.

Integration of 100% customer data in GIS and GIS-based mobile app for meter reading led to increase in billing and collection as shown in Table 1.

Table 1: Billing and collection efficiency

FY Year	2018-19	2019-20	2020-21	2021-22	2022-23
Bills raised (Rs. Crores)	31.53	38.28	34.54	38.06	39.06
Collection (Rs. Crores)	32.22	30.93	31.06	39.96	46.45

5. Status:

So far, six DMAs with 5000 HSCs are commissioned, with continuous 24×7 pressurised water supply. 300 km pipelines in 20 DMAs are commissioned with intermittent supply. These 20 DMAs will have a 24×7 water supply soon on receipt of additional bulk water from the new project to be commissioned shortly by Tamil Nadu Water Supply and Drainage Board (TWAD) Board.

6. Performance in the 24×7 WS commissioned DMAs.

Description	Name of DMAs				
	Jayaram Nagar	Revathy Layout	Cheran Nagar	New Madathur	Koilmedu
Month of Commissioning	Nov-21	May-22	Jun-22	Aug-22	Dec-22
No of HSCs	1,349	502	1,205	1,114	188
Population	15,758	4,947	11,838	8,815	1,699
* Supply (ML)	79.6	32.2	69	77.5	9
* Consumption (ML)	64.6	28	48.6	56.1	7.2
NRW in %	18.76%	13.05%	**29.49%	**27.58%	19.79%
LPCD	95.8	109	109.6	149.8	88.2
* Supply and Consumption for one billing cycle of 2 months ** Under Review					

The design requirement is minimum 7 m residual nodal pressure at the consumer ferrule point. But 86% of the project area will have a pressure range of 10 to 25 m.

7. Project key highlights:
   a) GIS-based map created using high-resolution satellite image
   b) Unique ID for each building parcel and door-to-door consumer survey and its integration with GIS
   c) Detailed water network and assets mapping using innovative GIS and digital technology
   d) GIS-based hydraulic modelling
   e) Digital dashboard for monitoring present intermittent water supply status and various other project Activities by deploying mobile applications.
   f) 100% of Water Meters are geo-tagged
   g) Dedicated Meter reading app developed with GIS data where all HSC data is mandated with photographs
   h) SCADA monitoring of entire water distribution with Central Control Room
   i) “E-Governance Initiative of the year by Govt. of India, 2022” for efficient digitalisation of water distribution system”. - Dec ’22, Guwahati

The project bagged a prestigious award of “E-Governance Initiative of the year by Govt. of India, 2022” for efficient digitalisation of water distribution system”. - Dec ’22, Guwahati

It also bagged another award of “GeoSmart India Excellence Award, 2022” for the GIS-based mobile meter reading APP. - Nov ’22, Hyderabad

7. Pune 24×7 Water Supply Project

(a) Introduction

The project of 24×7 pressurised water supply for the Pune Municipal Corporation (PMC) has been approved in 2015. Tenders have been invited accordingly by dividing the project into water tanks and transmission lines according to the WTPs.

Pune city is geographically divided into high and low areas. Therefore, water is available at high pressure and for a long time in some areas. And in some areas, there is very little water supply. About 40,000 non-residential households in the city are supplied with water through meters and the rest of the households are charged a lump sum water tax through property tax. The city currently has around 40% (NRW). There are many complaints like low pressure in the water supply system from time to time. The current water consumption is around 1,550 MLD which is more than CPHEEO norms. As the water lines in some parts of the city are old, complaints of polluted water are also being received.

For addressing above issues Pune Municipal Corporation has initiated this Project. The objectives of the Equitable Water Supply Project are as follows:

a) Thoroughly study all existing water supply systems and the entire water supply system, designing networks using up-to-date computer software.
b) Construct water tanks to increase storage capacity as required.
c) Lay necessary pressure pipes to supply water tanks.
d) Replace old water lines and install new ones between roads as required, dropping the old lines.
e) Achieve 100% replacement of water pipes and install smart meters with updated technology.
f) Construct new pumping stations and install updated automatic systems as required.

One of the components of the Project is storage tanks. Construction of 82 service tanks (ESR/GSR) is planned to increase water storage capacity as per CPHEEO norms.

This work was allotted to M/s Larson & Toubro Limited in the year 2016. The term of the said work is 30 months, and the said work has been extended for the third time till December 2023. So far, 44 out of 82 service tanks have been completed. The works of 20 tanks are in progress at various stages. Works of 18 tanks are pending due to non-availability of space elsewhere, which are legally sub-judicated cases, private space dispute, permission from Hon. Defence Department, etc. Necessary follow-up is being taken through the department for the said pending tanks. 50% of tender cost (Rs. 117.97 crores) for the work of tanks has been sanctioned by the government under Amrit Yojana. For the work of tanks, till now, Rs. 180.01 crore has been spent.

(b) Revamping of water supply Networks

Construction of pumping stations, installation of AMR meters and laying of new pipelines, etc., for waterworks are carried out under common water supply projects. A new estimate of Rs. 2307.10 crores has been prepared by PMC for the work. For the said work, tenders have been issued for WTPs jurisdiction and apart from that, two tenders have been released for the main waterworks. This work includes 3,18,847 AMR meters, 1550 km of distribution pipelines, 120 km of transmission lines. It also includes rehabilitation of existing WTP, updating and new construction of pumping station, improvement of water distribution system, repair of existing water tanks. It includes installation of PLC, SCADA, Smart Meter and maintenance repair for 10 years. Accordingly, five tenders were allotted to M/s. Larsen & Toubro and one to M/s Jain Irrigation. The work is underway at the site.

The works of laying water channels is the planned in 141 water supply zones. Out of the work of laying of 1550 km, the work of 830.51 km length has been completed. In transmission line works Package no. 6, a total 120 km transmission line works are planned. 73.483 km out of the total length of the work has been completed. So far, 119,746 AMR water meters have been installed, and Rs. 954.19 crores have been spent for the said project work.

According to project planning, the construction works of Parvati WTP and pumping station at Santosh Nagar, Katraj is in the final stage. After that, it is planned to do piping and electrical works. A total of five civic amenities centres are planned for the convenience of the citizens.

In the current scenario, out of 141 water supply zones of the Project, about 50 zones are given to contractor for further operation and maintenance works for the reduction of NRW. For detailed and authentic water audit, electromagnetic flowmeters are installed at locations from dam outlets to WTP inlets and ESR/GSR outlets to various DMA inlets. With the help of this information and data analytics, as well as machine-to-machine data transfer, realistic data is being provided for water audits.

(c) Bulk Meter at Entry of DMA

The electromagnetic bulk meters (Figure 1 and 2) are installed at the entry of the DMAs.

Figure 1: Bulk meters with sensor at entry of DMA

Figure 1: Bulk meters reader and transmitter

It is suggested to carry out the “zero pressure” test for this DMA for ensuring hydraulic discrete DMA. After that the DMA should be converted to 24×7 water supply. PMC is operationalising the DMA with 24×7 water supply as a demonstration zone and understand the nitty-gritty of the operation of the system. The survey on customer satisfaction and willingness to pay for the improved services is being carried out by PMC/L&T. Subsequently, the telescopic water tariff can be increased, and the learning may be implemented in other DMAs.

(d) Uniqueness of the Project

PMC in implementing installation of AMRs, one such meter is shown in Figure 1. It has the capacity of carrying out the reading of the measured consumption and to transmit the data via radio signals to be received by portable hand-held units. This will enable PMC to collect the meter readings with low cost with and the highest accuracy the estimate.

The main advantages related to the meters with the AMR capability are the following ones:

i. There is minimal need of human intervention in the collection of the water consumption readings and in the preparation of the water billing. This will reduce the possible mistakes or the possibility of human-made alteration to the true measured values.
ii. The proposed meters will have the capacity of not measuring the reversed flow, which is presently affecting the normal meter readings in the periods when the distribution pipes are emptying. This will increase the consumers' trust and confidence on the assessment of the true water consumption.
iii. The software which is currently provided with the AMR system may perform some automated computations that are useful for the water management either by the water utility company as well as for the consumers, such as to perform water balances at the DMA level and to compute statistics of the water consumption that may identify leakages or other anomalous behaviours.
iv. The relative ease and low cost for the assessment of the water consumption at each DMA level leads to carry out the water balance at any desired interval of time and at any level of the distribution system component. This is important in order to direct the water leakage detection where it is more needed and then to verify the effectiveness of the carried-out leakage repairs.

Figure 1: AMR meter at Ganesh Nagar

For getting consistency in AMR meter reading PMC is installing Fixed network gateways on certain locations. By this AMR meter can be read at desired interval from one point only. With the help of advance technologies, PMC is enabling systematic approach in resolving day-to-day needs in water supply.

So far, three DMAs have been converted to 24×7 PWSS.

8. A Case Study of 24×7 Water Supply of Nagpur

1. Brief of Project

The present population of Nagpur city is about 30 lakhs, and the area of jurisdiction is 218 sq. km. The existing average water supply is 650 MLD and average per capita supply is 217 LPCD. 'Pench-Khairy' dam and the perennial 'Kanhan' river supplies water to the five WTPs which supply water to the 10 water administrative zones of the city. There are 68 Elevated Service Reservoirs (ESRs) and 11 Ground Water Service Reservoirs (GSRs) having total storage capacity of 215.28 ML. The length of the feeder mains is 194 km, and the length of distribution pipe network is 4,300 km.

The water sources of Nagpur city are found adequate for 24×7 project. It was informed that the city gets water at the rate of 217 LPCD.

2. The Project

The Project is being implemented on a PPP basis. A global tender was invited. The bidder was expected to quote only a single rate against the volume billed multiplied by the commercial efficiency (Amount received/Amount billed). Apart from this, the private partner was expected to carry out the works under Initial Performance Improvement Project (IPIP) as per rates mentioned in the bill of quantity (BoQ) for which he would be paid separately. In addition, he was supposed to invest a certain amount on his own for regular repairs and renewals (R&R) such that the entire assets would be in good and running condition at the time of exit; designated at the end of 25 years from the date of agreement.

The tripartite agreement was signed between Nagpur Municipal Corporation (NMC), Nagpur Environmental Services Ltd (NESL), and M/s Orange City Water Private limited, a joint venture between M/s Veolia India Private Limited and M/s Vishwaraj Environmental Private Limited in the month of March 2012.

3. Special Purpose Vehicle

Nagpur Environmental Services Ltd (NESL) is the SPV established by the NMC. While the operator is given the necessary access to enable smooth discharge of its duties, the sovereign rights of tariff setting, allotment of new connections, and disconnections is reserved with the NMC. Also, the ownership of all the assets remains with Nagpur Corporation.

4. Financial

The funding pattern is shown in Table 1.

Table 1: Funding Pattern

Total Cost of Project (Rs. Crores)	GoI Share at 50% of Total Cost (Rs. Crores)	GoM share at 20% of Total Cost (Rs. Crores)	NMC Share at 30% of Total Cost (Rs. Crores)	Remarks

387.86	194.93	77.57	116.358	NMC's share of Rs. 116.358 crore is to be invested by Private Partner.

The works included in this project are:

(a) *Capital Cost:* Replacement of pipelines (inside and outside slums) of 688 km and 321,327 number of HSCs (inside and outside slums). The works also included rehabilitation of MBR, ESR, and GSR which are 20 in numbers. It also includes laying a 2000 mm diameter of length 2.312 km. The lump sum jobs like SCADA are also included. The cost of all these works summed up to Rs. 387.86 crores. The clause of escalation was applicable to the same.

(b) *O&M Cost:* NMC collects about Rs. 211 crores as user charges. OCW is expected to make expenditure on renewal and replacement every year as mentioned above.

5. Status of Works

OCW claims that 33 out of 68 OZs (except slum areas) are presently receiving water on 24×7 basis. OZ is defined as the command area of each individual service tank. The work was expected to be completed in another five years. The progress of works is shown in Table 2.

Table 2: Progress of works

Sr. No.	Particulars	Unit	Quantity to be executed	Quantity Executed	Quantity remained to be executed
1	Replacement of pipeline (inside and outside slums)	Km	688	739	-
2	HSCs (inside and outside slums)	Nos	321327	2,88,352	32,059
3	P/L/L/J MS pipeline 2000mm dia	Km	2.312	2.312	-
4	Rehab of ESR, GSR, MBR	Nos	20	19	1

4. Demo DMA

The ESR in the demo DMA is at the 'Triveni Nagar' (Figure 1). This OZ is observed to be too large.

Figure 1: ESR of Triveni OZ

Figure 2: Consumer's underground tank

It is observed that each consumer has an underground tank (Figure 2). Consumer meters were provided to all the consumers (Figure 3).

Figure 3: Consumer's meter

Figure 4: Bulk Flow Meter and pressure gauge at entry of DMA of Triveni OZ

All the consumers informed that they get 24×7 continuous water, but water is not reaching to even to the first floor.

Bulk Meter at Entry of DMA: The Bulk Flow Meter is found installed (Figure 4) and pressure gauge at entry of DMA of Triveni OZ.

Problem of the leaking underground tanks:

The underground sumps/tanks of customers are leaking that could be one of the reasons of high NRW in the city.

Lessons Learnt:

1) Modern technique of GIS mapping is excellently made in the city.
2) Consumer survey of entire city has been conducted. All the customers are geo-tagged for water billing.
3) The scheme has been planned with the concept of DMAs.

Above three merits of the scheme are noted. But there are some odd things observed in this project which are summarised as below:

(a) The scheme has been designed for residual pressure of just 2 m. Therefore, the extent of the OZs is too big. Hence, it is recommended that for every 24×7 pressurised water supply projects, the boundary of the OZs should be optimum which means it should not be too large to avoid tank getting empty and at the same time it should not be too less to avoid overflowing of the service tanks.
(b) The pipe replacement proposed in the scheme is just 20% which is found insufficient to lower down the present NRW of 45%. The survey of existing infrastructure and its GIS mapping must be carried out to arrive at proper percentage of existing pipe replacement.

Solution:

In spite of low residual pressure, the ULB is able to convert 33 out of 68 OZs to 24×7 water supply. The staging height of most of the ESRs in the city is 20 m. Hence, it is possible to increase the residual nodal pressures to at least more than 7 m though the network is designed for 2 m. This can be done by slowly and marginally opening of the isolation valve installed on the outlet of the ESR.

Thus, in such cases, the concept of gradual conversion to 24×7 water supply by increasing residual pressure marginally by an increment of 1m and again subsequently by the increments of 1 m shall be useful to achieve 24×7 water supply.

Technical advancement for efficient implementation of the project:

GIS Mapping: GIS is implemented to facilitate operation and maintenance and CAPEX activities, to carry out business analysis and decision making all water assets like WTPs, pumping stations, Feeder mains, MBRs, ESRs, GSRs and distribution network are digitized along with valuable asset information. Special emphasis is given to mark the location and information of each and every consumer and each and every pipeline laid. Sample map is shown in Figure 5.

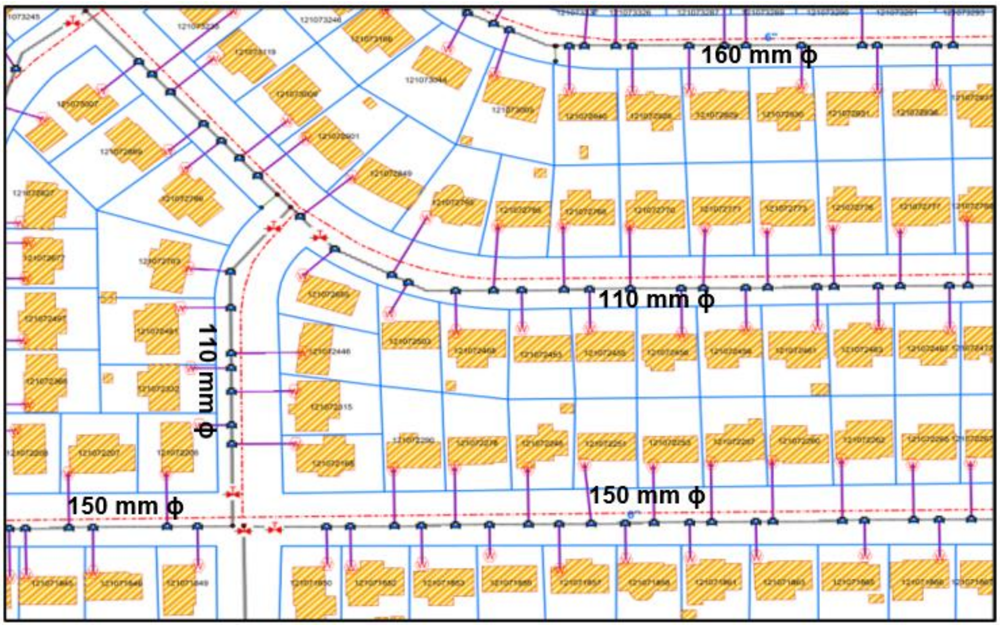


Figure 5: GIS: Integration of Water Network with Consumer database

SCADA: SCADA at OCW is based on open protocol platform as Modbus for complete multi-platform integration. OCW SCADA communicates with remote locations (ESR & GSR) via GPRS system and Radio RF System by local RTU panels (Figure 6). SCADA system is implemented for water management at entire city i.e. monitoring of water quality, flow, pressure, ESR levels and pressure at various monitoring points.

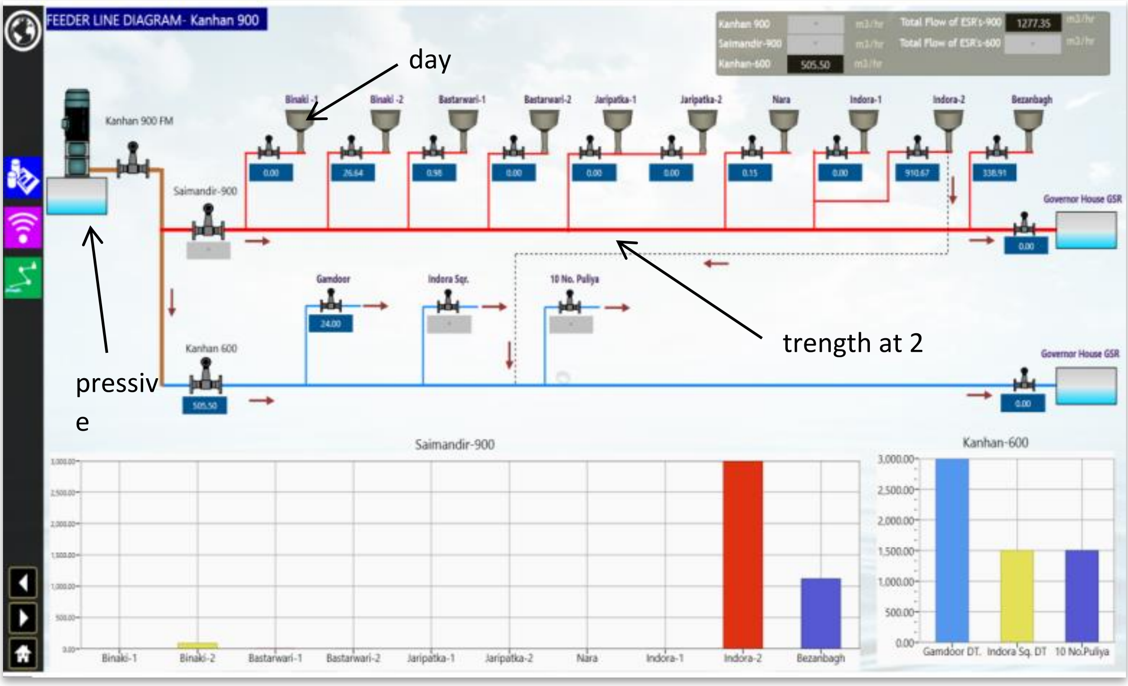


Figure 6: Feeder Monitoring through SCADA

SAP billing system: OCW takes photo meter reading by app and loads to SAP directly and bill through SAP ISU system for 3.5 lakh consumers. It is the heart of revenue generation. ECS, NEFT, UPI facilities are added tool for revenue collection along with physical collection counter across the city. SAP generated payment receipts provided to the consumer as an acknowledgement.

24x7 Complaint Management System: This tool is developed keeping in view a customer centric approach. Consumers can use platforms like OCW toll free number, Customer Care Centre or Water House to raise any issue related to water supply. The raised issues are registered with unique number 1800 266 9899 and are assigned to concerned field staff for resolution. Once the issue is resolved, field order form is generated and cross verified for consumer satisfaction. OCW engaged people in customer care centre and work for redressal of complaint round the clock.

GPS monitoring of tanker movement: For efficient and effective management of water supply through tankers, tanker movements are monitored through GPS system to ensure tanker delivery to needy peoples.

Tank Cleaning System: OCW has developed in-house tank cleaning system (Figure 7) for elevated and ground service reservoirs. Before taking over, the cleaning of these tanks were either neglected or done through inappropriate means like the services of a local plumber or sweepers were engaged. The core highlight of the system is high pressure water jet. The in house built system consists of high pressure pump of 150 Bar capacity, a Low pressure pump for flushing & rinsing and a chemical dosing pump. Now OCW is carrying out cleaning of every ESRs at scheduled intervals.

Figure 7: Tank Cleaning System

Social Innovations: OCW developed various social innovations to smoothen the process of citizen on-boarding. To achieve these innovations, OCW established a dedicated Social Welfare Team (SWT) with specialist professionals to on board the ultimate beneficiaries of the project. The team communicates (Figure 8) with the masses in three phases- before, during and after the rehabilitation work and 24x7 pressurised water supply system implementation.

Figure 8: Social Welfare Team

SWT team extended its service through 470 water friends from dynamic energetic citizens. Also, OCW extend awareness towards water conservation and efficient water use at various school and colleges at city along with mohalla meeting at various important locations.

9. A Case Study of 24×7 Water Supply of Visakhapatnam

1. Brief of Project

The city of Visakhapatnam is the largest city in the state of Andhra Pradesh. The city is on the east coast of India and is a major industrial hub, port, tourism, and headquarters of Eastern Naval Command. The total area covers 681.96 sq. km with a population of 2.40 million. The city is ranked 4th in Swachh Survekshan under "Million plus category" and awarded "Water+" certification. The city received award on "Optimum and Efficient Utilisation of Water Resources" from Central Board of Irrigation and Power for the year 2022. The ULB decided to embark on 24×7 PWSS in one part of the city which is called as "Gajuwaka Visakhapatnam Municipal Corporation (GVMC)."

2. The Project Area

The water supply service area of GVMC is spread across 620 sq. km., which includes Gajuwaka and adjoining peripheral areas. The service area is divided into four major water supply zones: namely, old city area, central zone area, Gajuwaka area and 32 peripherals areas. The central zone area is further divided into NE Sector, SW Sector, and NW Sector. The area of the present project falls in 'NW Sector of Central zone area' and consists of 32 blocks. Each block on an average has three to four DMAs. The population of the NW Sector, as per Census 2011, is 2,67,334 and it is spread over an area of 40.389 sq. km. GVMC area is divided into 98 municipal wards of which 21 wards fall under the NW Sector. Key information of the Project is shown in Table 1.

Table 1: Key information of the Project

Location	NW sector in Central City of GVMC
Area	40.39 Sq. Km.
Total no of HSCs	48,000 Nos
Served Population	3.0 Lakhs
Ensured Quantity per Capita	135 LPCD
Assessed Water requirement for Project	42 MLD
Total No of Blocks	32 Nos
Total NO of GLSR's	19 Nos
Total No of ELSR's	10 Nos
Total capacity of service reservoirs	28,855 Kl (68% of total Required Qty)
Total No of DMA's	149 Nos
Capex of the project	Rs. 398.27 Cr (Including GST)
Opex of the project	Rs. 48.72 Cr (Including GST)
O&M Period	10 Years from the date of commissioning
Funding Agency	ADB: 78.50% + State of AP: 21.50%
Project Management Unit	VCICDP
Project Implementation Unit	GVMC, Visakhapatnam
Contract Agency	NCC Limited, Hyderabad

3. Objectives of the Project:

The project has following objectives:

a) reduction of non-revenue water (NRW) from 38% to 10%);
b) supply of 135 LPCD water for 24 hours with 12 m head at consumer tap;
c) HSCs with advanced metering infrastructure (AMI) meters;
d) computerised monitoring - SCADA for better management and efficient monitoring.

4. Service improvement plan (SIP):

Before implementing the project, the following activities related to service improvement plan are carried out:

a) Complete door-to-door consumer survey to assess in actual population, the footprints and property through satellite imaging
b) Digitisation of all the footprints, roads, water bodies, electricity poles, water supply and other infrastructure components in the satellite image
c) To geo-code the household survey in the digitised satellite image
d) Conducting Ground Penetrating Radar Survey (GPRS) for detection of underground utilities up to 1.5 m depth and mark on GIS based maps by linking with geo referenced points
e) Installation of flowmeters/domestic meters and for assessing of initial NRW

5. Salient Features of the Project:

They are as below:

a) Preparation and implementation of Service Improvement Plan (SIP)
b) Distribution Network Improvement (DNI) on DMA for NRW
c) Reduction and efficiency improvement in continuous water supply
d) HSCs with Advanced Metering Infrastructure (AMI)
e) 24×7 water supply ensuring continuous pressure with 12 m head
f) Computerised monitoring mechanism through integration with SCADA for better management
g) Consumer friendly ‘Grievance Redressal Mechanism’ during ‘Operation & Maintenance’

6. Proposed in Service Improvement Plan of the Project:

Following Service Improvement Plan are carried out:

(a) Survey and mapping: Survey has been carried out for existing network & NRW study, topography survey, GPRS, consumer survey and GIS mapping
(b) Network modelling: Activities are to prepare hydraulic modelling and network design and DMA formation
(c) Smart controls: Smart controls are made for pumping and gravity mains, service reservoirs, distribution network, pressure management, HSCs, and water quality management
(d) Smart operations: Activities are - SCADA and AMI, bill generation and collection, consumer relation monitoring centre

Assessment of NRW in Existing System:

NRW is assessed as below:

a) Total House assessments surveyed : 73,046 Nos.
b) Total No of Tap assessments surveyed : 44,031 Nos.

c) Total Quantity of Water Supplied : 41,664 Kl
d) Total Consumption recorded at HSC's : 27,155 Kl
e) Total losses (Service reservoir to HSC's) : 14,509 Kl
f) Total NRW% : 34.82%

7. Automation

(a) Automation through SCADA: Architecture for management of 24×7 Water supply through SCADA is shown in Figure 1. SCADA is proposed at the central station to operate and monitor entire 24×7 command area in real time. It is used for data analytics for auto pump operations and data analytics for NRW assessment.

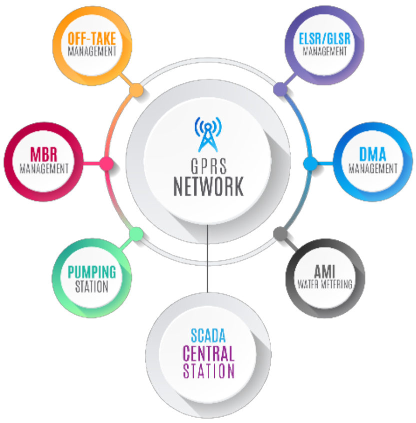


Figure 1: Architecture for Management of 24×7 Water supply through SCADA

Figure 2: Human machine interface of SCADA

(b) Proposed Each DMA Inlet Managements

The arrangement at every DMA is shown in Figure 3. Every customer's meter is provided with data communication chip which send metering data to the LORA gateway placed at entry point of each DMA.

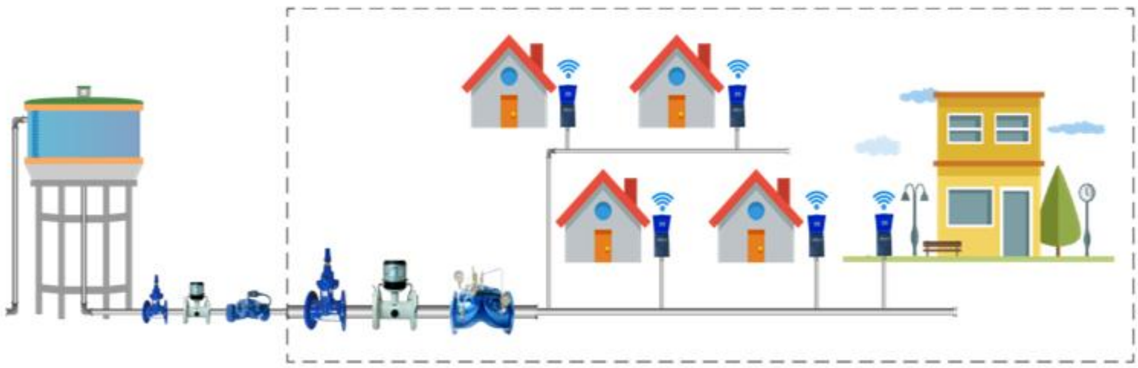


Figure 3: Arrangement at every DMA

At each DMA following is provided:

i. Hydrodynamic and electric-operated control valves with 17 W/24 V DC solenoid control to operate from local/remote on each inlet of the individual DMA to control pressure to ensure constant downstream pressure and close automatically as and when there is no demand.

ii. Self-powered ultrasonic flowmeter to measure (Figure 4) the accurate discharge of each and every individual DMA to measure the flow and volume to assess and conduct NRW Audit.

iii. RTU control panel (5) with solar-based power to connect all instruments to operate auto/manual, data compiling and synchronise with a central server for real-time operation and monitoring.

Figure 4: Flow and pressure measurement at entry point of each DMA

Figure 5: RTU at entry point of each DMA

8. HSCs and Communication system

Overall communication system for HSCs to central system is shown in Figure 6 with AMI and fixed network.

HSCs are proposed have following details:

1. HSC is given using PE 100 HDPE pipes with 15, 20 and 25 mm to be suitable for a 24×7 water supply system
2. Provided electrofusion saddle joints for taking the HSC connection from distribution main
3. Providing ball valve for emergency operation by the consumer to avoid wastage of water
4. Proposed AMI with IP 68 protected self-powered ultrasonic flowmeter with LoRa communication in pre-fixed intervals to send the data for usage of each HSC
5. Proposed PE Box with locking screw to ensure flowmeter safety from the environment and ensure data transmission through LoRa network

6. Adopted fixed network by providing wireless gateways on high-raised public/private buildings for collecting the data from AMI meters through LoRa and sending data to SCADA through the GPRS network for data analytics, water audit, and reporting
7. Providing solar power supply to the wireless gateways with power backup for uninterrupted power supply.
8. Adopting daily monitoring, collecting data from AMI meter for better management of NRW

Figure 6: Overall communication system

8. Financials: Component wise Expenditure is shown in Table 2.

Table 2: Component wise expenditure

S.No.	Component	Value (Rs. Cr.)	Share %
1	Pumping and Gravity Main	44.6553	9.99%
2	Transmission Main - 1	26.19	5.86%
3	Transmission Main - 2	17.34	3.88%
4	Reservoir	19.53	4.37%
5	Distribution Pipes	116.39	26.04%
6	Pumps and Machinery	17.56	3.93%
7	HSC	99.27	22.21%
8	Civil Works	2.01	0.45%
9	SCADA	22.35	5.00%
10	Road Restoration	25.47	5.70%
11	Miscellaneous Items	7.46	1.67%
12	O&M/10 years	48.72	10.90%
	GRAND TOTAL	**447**	**100%**

Per Capita Cost of Project: It is mentioned in Table 3.

Table 3: Capita cost of project

1	Total Population can be served	300,000 Members
2	Total CAPAEX of the project	Rs. 4,00,00,00,000/-
3	Per Capita Expenditure	Rs. 13,276/-

Tarif Structure of 24×7 water supply: It is shown in Table 4.

Table 4: Tarif Structure of 24 × 7 Water Supply

1	0 to 5 KL	Rs. 10/-
2	Above 5 KL	Rs. 10/- per KL
3	Minimum Billing (****Whichever is Higher)***	Rs. 125/-

NRW Status: Present revenue advantage due to reduction in NRW is shown in Table 5.

Table 5: Observed NRW

S No	Year	Block 6 2 DMA's	Block 7 2 DMA's	Block10 4 DMA's	Block 12 4 DMA's	Block 13 11 DMA's	Block 14 01 DMA	**Remarks**
A	Before Implementation of 24×7							
1	Initial NRW	18.54%	23.81%	41.19%	41.51%	67.01%	41.81%	**Before 24×7**
		38.97%						
Population		2031	2098	4786	8369	8013	4493	
B	After Implementation of 24×7							
1	2019	6.54%	3.93%	8.45%				**After 24×7**
2	2020	0.31%	1.06%	5.16%				
3	2021	1.75%	2.00%	5.85%				
4	2022	4.40%	5.72%	6.21%		4.47%		
5	2023	5.74%	5.46%	7.56%	7.52%	6.47%	8.20%	
Average		3.75%	3.63%	6.65%	7.52%	5.47%	8.20%	
Average		5.87%						

10. Indi 24×7 Water Supply System

1. About Indi town

Indi is a fast-growing town situated in the Vijayapur District and about 50 km from Vijayapur City, i.e., district headquarters. Indi Town is located at latitude and longitude of 17°17′40″N 75°97′56‴E. It has an average elevation of 465 m above MSL. The Indi Town has 23 wards and equal number of councillors. The population of the town as per 2011 census is 38,329 and the present population is about 48,000.

Bheema River is the source of the 24×7 water supply scheme of the town which is Near Takali-Dhulakhed barrage. This source is implemented by KUWS&DB, and it is shown in Figure 1.

Figure 1: Source Bheema River

This project comprises of infrastructure development along with reforms in water sector. The proposed project is primarily focused on providing assured bulk water supply from perennial surface water and also improving the water distribution and service delivery to the customers by providing continuous pressurised (24×7) water supply service concept.

2. Assets created for 24×7 Pressurised Water Supply System

The 24×7 water supply scheme has been approved in 2018. The project has been implemented and commissioned in the year 2020. Following are the components of the project:

1. Headworks: It has 10 m diameter Jackwell cum pump house including 4 m diameter intake well and 370 HP raw water pumping machineries
2. There are two rows of connecting mains with 700 mm diameter RCC NP-3 class with a length of 125 m
3. 11 KV express feeder main from Dhulakhed sub-station - 3.06 km
4. 508 mm diameter 5.6 mm thick MS raw water rising main from Headworks near Takali-Dhulakahed barrage to WTP at Indi town - 33.65 km

5. 5.00ML in 20 hours WTP along with 5 LL capacity sump cum pump house and 60 HP centrifugal and 20 HP submersible pump set at WTP
6. Renovation of existing 5.00 MLD WTP with 60 HP submersible pump sets
7. 250 mm diameter and 300mm diameter DI Feeder Mains from WTP in Indi Town - 2.11 km
8. 110 mm and 160 mm diameter HDPE feeder mains from WTP in Indi town - 7.98 km.
9. 10.00 Lakh litres capacity Shaft type RCC ESR on 15 m staging at Zone 4 KEB backside in Indi town (1 No of 5.00 Lakh litres cap, 9.08 Lakh litres capacity, 1 No of 10 Lakh litres existing OHTs)
10. 1.00 lakh-litres capacity column type RCC ESR on 15 m staging at Satpur
11. DI (250 mm diameter) and HDPE (75 mm to 200 mm diameter) water distribution network and HSCs (10,000 Nos). Zone - 1, 2, 3, 4, Satpur and Dhanashetty tanda in Indi Town - 185.50 km.

Various components of the scheme are shown in Figure 2.

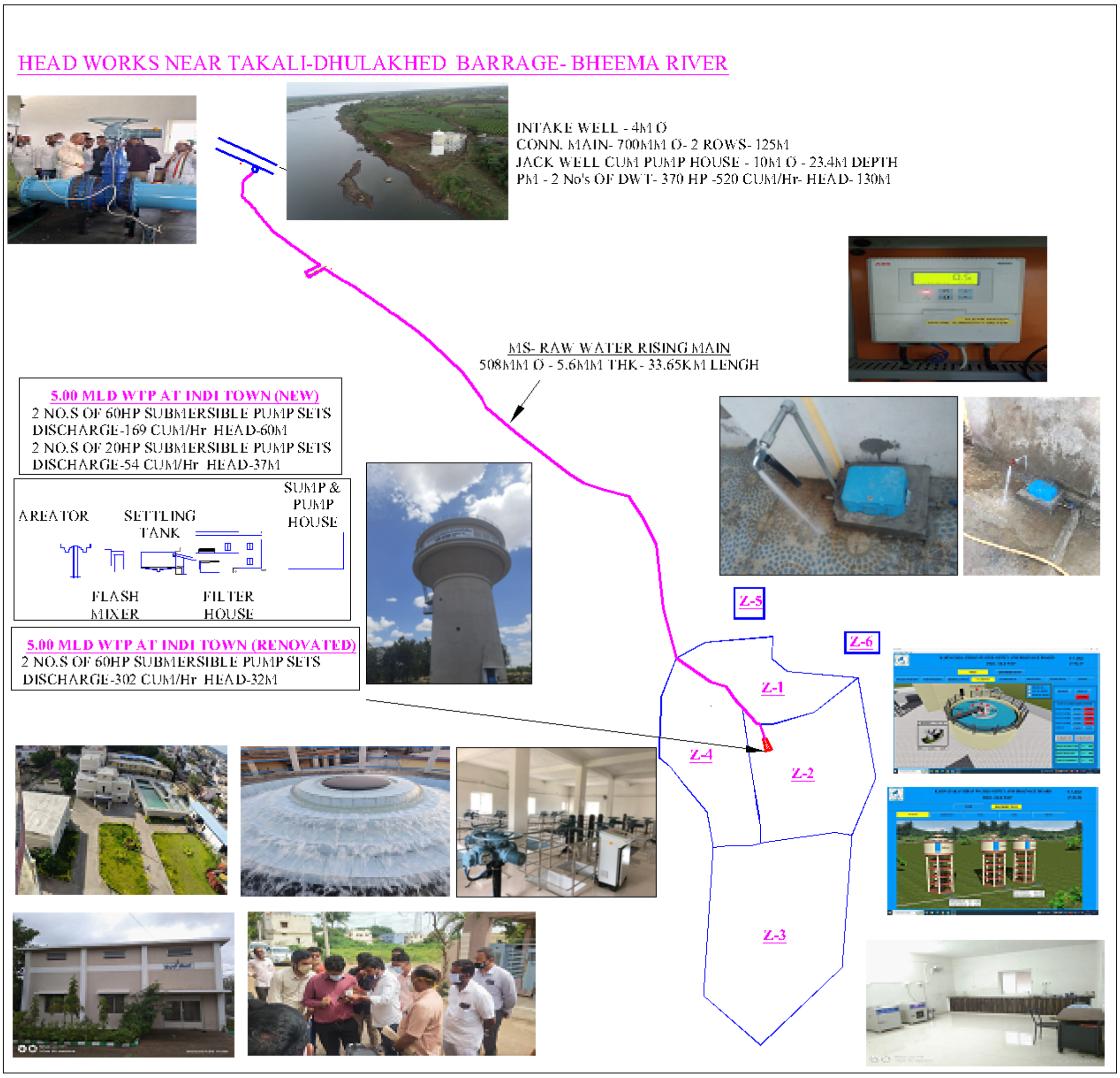


Figure 2: Various components of the scheme.

3. Improvements

Improvements in the SLBs after the implementation of the project are shown in Table 1:

Table 1: Improvements in the SLBs

S. No.	Indicator	Unit	Baseline Before Project	After Project
1	Coverage	%	60%	100%
2	Per Capita Supply	LPCD	135	135
3	Metering	%	Nil	1
4	NRW	%	40%	<15%
5	Continuity of Supply	Hours/day	4 to 6 hrs. once in 8-10 days	24 hrs.
6	Quality	% water sample meeting water standards	No data	100%
7	Efficiency in Redressal of Customer Complaints	%	50%	100%
8	Cost Recovery of water supply services	%	60%	100%
9	Efficiency in Collection of Charges	%	70%	100%
10	Residual pressure at tap	m	In-equitable pressure	Minimum 7m
11	Residual chlorine at tap	ppm	-	< 0.1 to 0.2 ppm

4. Key parameters adopted to achieve “24×7 WATER SUPPLY” mission:

They are as below:

1. Real-time monitoring and control of raw water pumps at headworks, operations of automated 5.0 MLD cap WTP, pure water pumps at WTP, inlet and outlet of reservoirs
2. Real-time monitoring of flow from headworks, WTP and to reservoirs. Also, outflow data from reservoirs
3. Real-time monitoring of quality parameters (turbidity, pH, chlorine) at WTP and also at six DMAs
4. Real-time monitoring of all five reservoirs levels
5. The entire system operations from source to sink; real-time monitoring is based on cloud/web
6. NRW is ranging from 7% to 15%
7. Improved collection compares to demand
8. Customer redressal system through control room

11. Thirthahalli 24×7 Water Supply System

1. About Thirthahalli town

Thirthahalli Town is a Taluka headquarter in Shimoga District. The town comes under ULB having town Panchayath office for local administration. The population of the town as per 2011 Census is 14,537, and present population is about 15,000. The National Highway No. 13 connects Solapur and Mangalore and it passes through Thirthahalli Town. River Tunga is flowing with in town limit. The town is having surface water supply from Tunga River source.

The town is situated on left bank of River Tunga and on the other side river, Kushavathi is flowing as tributary to river Tunga. Thirthahalli town is situated at elevation of 653 m above MSL. Configuration of the town is 130°45′ latitude and 75° 15′ longitude. Tunga river and Kushavathi tributary is passing within the town limit. Thirthahalli has a moderate climate. The average rainfall to the city is about 2,128.00 mm.

The town is at a hilly terrain surrounded by Western Ghats. The topography of the town consists of many hillocks with high ridges and valleys; granite and sand quarry are situated in and around the town.

2. Assets created for 24×7 Water Supply System:

This 24×7 water supply scheme is sanctioned for Rs. 10.11 Crores. Source of the Thirthahalli town is river Tunga. The scheme has been implemented and has been providing 24×7 water supply to the city since 2018.

3. Components of the project

Following are the components of the project:

- Headworks: 6.00 m size Jackwell with 6.00 m size pump house including 3.0 m diameter Intake well and 40 HP DWVT raw water pumping machineries
- 273 mm diameter MS raw water rising main from headworks near Balebylu to WTP inside Thirthahalli town - 0.11 km
- 4.54 ML in 20 hours WTP along with 75 HP centrifugal pump set and 10 HP capacity submersible pump set at Balebylu, Thirthahalli town.
- 90 mm diameter to 250 mm diameter MS/DI/PVC pure water rising main - 6.21 km
- 10.0 lakh litres (LL) Cap. RCC GLSR 1 at Kolikalgudda
- 2.5 LL Cap. RCC GLSR 2 at Kolikalgudda
- 1.5 LL Cap. RCC shaft type OHT on 12.0 m Staging at Soppugudda old
- 1.5 LL Cap. RCC shaft type OHT on 12.0 m Staging at Soppugudda old
- 2.5 LL Cap. RCC OHT on 9.0 m Staging at J.C. Hospital
- 1.0 LL Cap. RCC GLSR at Kuruvalli
- 5.0 LL Cap. RCC OHT on 15.0 m staging at Balebylu
- 0.50 LL Cap. RCC GLSR at Gandhinagar
- 1.0 LL Cap. RCC OHT 15.0 m staging at Bettamakki
- HDPE (75 mm to 315 mm diameter) water distribution network and HSCs. Zone - 1 to 9 in Thirthahalli town - 64.18 km

Various components of the project are shown in Figure 1.

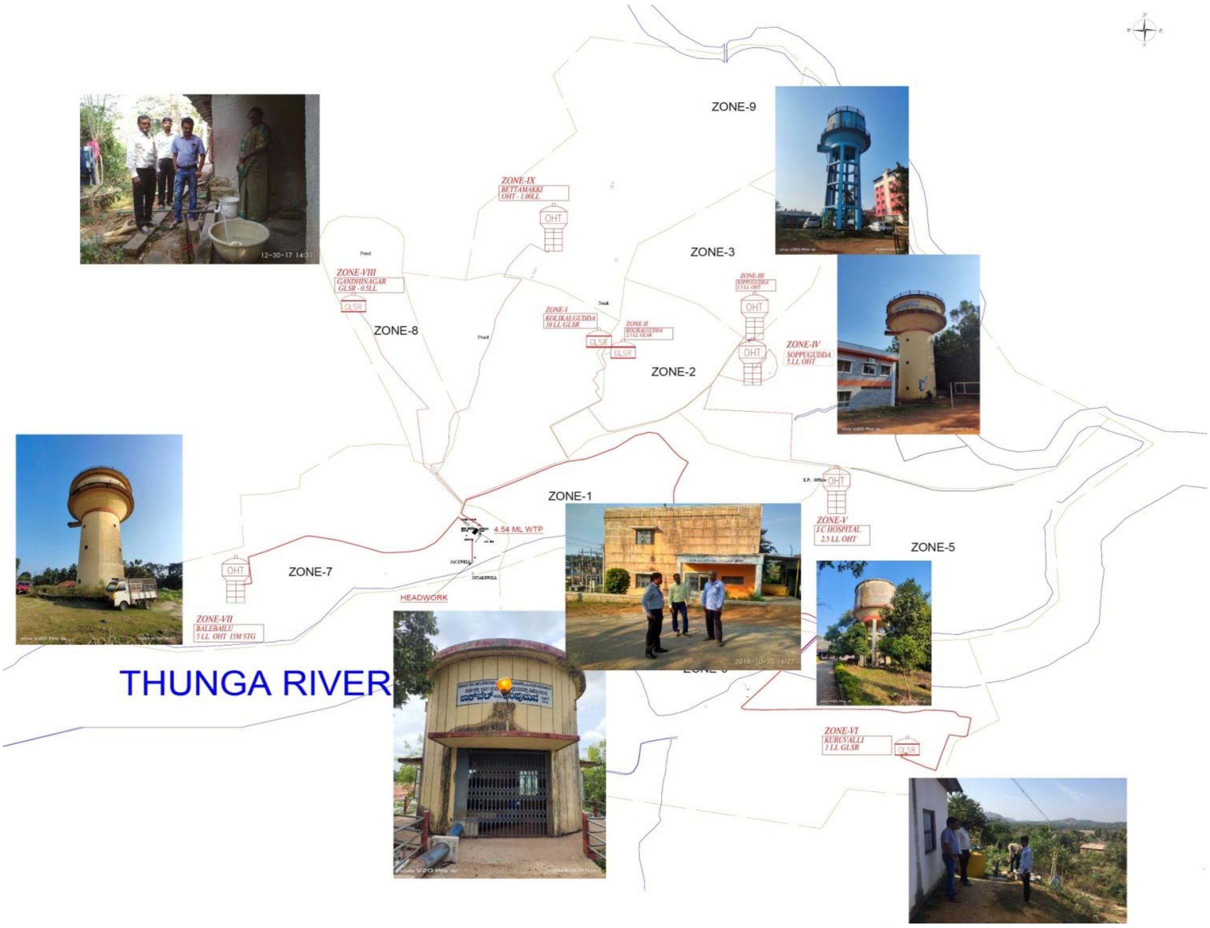


Figure 1: Various components of the project

4. Improvements

Improvements in the SLBs after the implementation of the project are shown in Table 1.

Table 1: Improvements in the Service Level Benchmarks

SN	Indicator	Unit	Baseline Before Project	After Project
1	Coverage	%	75%	100%
2	Per Capita Supply	LPCD	70	135
3	Metering	%	Nil	100%
4	NRW	%	20%	<5%
5	Continuity of Supply	Hours/day	3 to 4 hrs. daily	24 hrs.
6	Quality	% water sample meeting water standards	No data	100%
7	Efficiency in Redressal of Customer Complaints	%	50%	100%
8	Cost Recovery of water supply services	%	60%	100%

SN	Indicator	Unit	Baseline Before Project	After Project
9	Efficiency in Collection of Charges	%	70%	100%
10	Residual pressure at tap	m	In-equitable pressure	Minimum 7 m
11	Residual chlorine at tap	ppm	-	< 0.1 to 0.2 ppm

12. Shirpur 24×7 Water Supply System

1. About Shirpur town

Shirpur is an important and fast-growing town (B-Class) in the draught prone Dhule district of Maharashtra. It is located on National Highway 3, which runs from Agra in Uttar Pradesh to Mumbai, Maharashtra. The Arunawati River flows through the city. Shirpur is just 50 kilometres away from the city of Dhule. It also houses Asia's largest and India's first gold refinery. The average rainfall is 722 mm. The city administration envisaged 24/7 continuous drinking water supply scheme. The project was sanctioned by GoI, for a cost of Rs. 30.77 Crores.

Water is pumped from the headworks at the upstream side of the pick-up weir across the Arunavati river to the WTP. From pure water sump of the WTP, treated water is pumped to the four existing ESRs at Bhaskar Bapu, Khandesh Package, Police line, and APMC and then it is supplied to the respective distribution systems (Figure 1).

2. Hydraulic Model: Advanced and powerful techniques of GIS mapping and hydraulic modelling have been used in the present study. Using these techniques, a hydraulic model simulating the water supply system, right from the source to the consumers in distribution system, has been prepared. The scheme was commissioned in 2017.

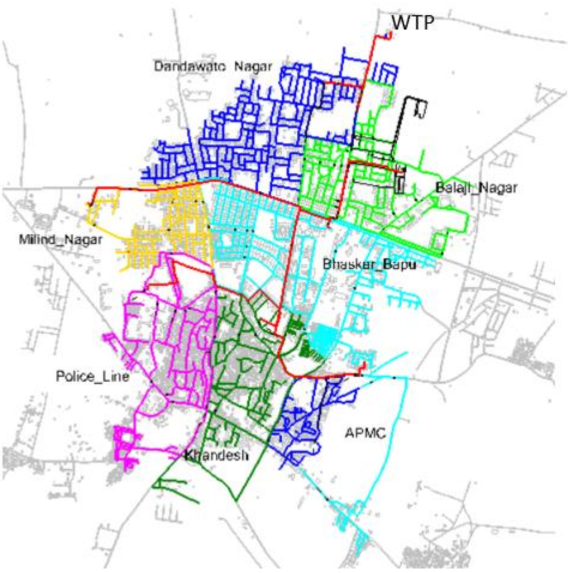


Figure 1: Distribution network of Shirpur town

Figure 2: AMR walk by system

3. Smart Water Meters: Smart Water meters have proven to make a difference in terms of conservation and speedy redressal of leaks leading to improved economical sustainability. Shirpur is one of the first cities to have 100% smart metering coverage with the installation of over 13,500 AMR smart water meters with walk by system (Figure 2). The officials in Shirpur have reported to have saved over four million litres of water and have a 100% billing efficiency. Furthermore, the city officials also have claimed to provide 24/7 water supply to all the residents in spite of it being in a drought prone region. In addition, it has reduced water thefts and improved redressal of leaks and NRW.

4. Learning from the Scheme

The scheme was commissioned with 24×7 pressurised water supply. Water reached even to the third floor of the consumers. The scheme was operated for two years. But after that, the residents have not paid water charges fully. Hence, the municipality decreased supply hours from 24 to 10 hours a day.

Hence, vigorous information, education, and communication, is required to resume 24×7 pressurised water supply.

13. Status of 24x7 Water Supply Projects in India

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
1	Alnavar	Karnataka	Providing Water Supply Scheme to Alnavar Town with Kaali River as source.	Alnavar town- 23,078 and 14 Enroute villages - 21,570 Total = 44,648 for the prospective year 2045	21,480 (Present Population)	10.59 sq. km	10.59 sq. km	Commissioned on 30.06.2022	2 years	Rs. 4190.00 lakhs (Kaali River source is located at 35Km away)	Rs. 6100.00 lakhs	RW & TW rising mains - 35.0 Km Distribution network - 57.0 Total = 92.0 Km	100%	100 %	Rs. 13,662/-

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
2	Chandigarh	Chandigarh	24 × 7 Water Supply PAN City	12.60 Lakh	12.60 Lakh	114 sq. km	114 sq. km	1. DPR approved 2. Tender for LTTA received and under evaluation 3. RFQ for selection of DBO Floated and reply to Queries of bidders are being prepared and Bids will be received by end of this month.	4 Year (2023-2027)	512 Crore	512 Crore	1350	22%	100% Smart Meters	Rs. 4063

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
3	Coimbatore	Tamil Nadu	Coimbatore 24×7 Water Supply Project	18,86,500	12,34,906	254 sq. Km	105.6 sq. km (Pre-2011 expansion area)	Ongoing	Study - 1 Year EPC - 4 Years, O&M - 25 Years (Including EPC 4 Years)	Not part of the project	646.71 Cr	1820 Km (Feeder main - 75 km + Distribution main 1745 km)	89%	100%	5236.91
4	Guwahati	Assam	Southeast Guwahati Water Supply Project (SEGWSP)	1279156	335720	216 km^2	71 km^2	Phase I - Completed. Phase II - DPR Approved.	5 Years (2024-2029)	NA	882.67 Cr	Transmission Mains - 27.5 Km Distribution - 553 Km	Nil	Nil	26291
5	Nagpur	Maharashtra	Rehabilitation Plan to Implement 24 × 7	31.4 Lakhs (March 2023)	29.75 Lakhs (March 2023)	217 sq. km	58.67 sq. km	Ongoing	10 years	0	Rs. 387.86 Crores	4332 KM	35% (of existing 1600 km at time of	100%	1303.73 Rs.

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Water Supply, for Nagpur City										commencement)		
6	Bhubaneswar	Odisha	24×7 & DFT to different areas of Bhubaneswar Municipality.	1221100	1221100	422	422	Ongoing	2 Years	NIL	641.00 Cr.	1378.05	10%	100	5249.37
7	Jatni	Odisha	24×7 & DFT for Jatni Municipality.	56630	56630	17.00	17.00	Ongoing	2 Years	NIL	12.50 Cr.	84.15	3%	100	2207.31

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
8	Khurda	Odisha	24×7 & DFT for Khrodha Municipality.	55900	55900	49.61	49.61	Ongoing	2 Years	NIL	15.40 Cr.	165.37	4%	100	2754.92
9	Puri	Odisha	24×7 Water supply to Puri Town Under DFT Mission.	266595	266595	18.54	18.54	Commissioned	2 Years	NIL	22.88 Cr.	335.88	8%	100	858.23
10	Nimapara	Odisha	Implementation of 24×7 W/S to Nimapara NAC.	22211	22211	14.07	14.07	Ongoing	2 Years	NIL	15.53 Cr.	83.31	10%	100	6992.03

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
11	Cuttack	Odisha	24×7 & DFT to different areas of Cuttack Municipality	696000	696000	192.50	192.50	Ongoing	2 Years	NIL	102.31 Cr.	1021.4	12%	100	1469.97
12	Vyasanagar	Odisha	Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap quality to	86000	86000	35.50	35.50	Ongoing	2 Years	Not worked out separately	99.43 Cr.	228.91	14%	100	26617.44

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Vyasanagar on Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years (Package-I)												

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
13	Berhampur	Odisha	Conversion of intermittent Water supply of Berhampur town to 24×7 Water supply under DFT Mission	448370	448370	79.80	79.80	Commissioned	2 Years	NIL	43.83 Cr.	324.67	1%	100	977.54
14	Gopalpur	Odisha	Conversion of Intermittent water supply to 24×7 water	10718	10718	4.10	4.1	Commissioned	2 Years	NIL	6.04 Cr.	18.62	5%	100	9740.62

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			supply to Gopalpur Town.												
15			Improvement with 24×7 Water Supply to Gopalpur NAC						2 Years		4.40 Cr.				
16	Hinjilicut	Odisha	Implementation of 24×7 water supply to Hinjilicut Municipality	30050	30050	11.81	11.81	Ongoing	2 Years	Not worked out separately	62.29 Cr.	51.45	11%	100	20728.79

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
17	Rajgangpur	Odisha	Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap quality to Rajgangpur on Engineering, Procurement & Construction (EPC)	62008	62008	26.99	26.99	Ongoing	2 Years	NIL	21.08 Cr.	133.05	6%	100	3399.56

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Contract Including Operation & Maintenance for a Period of Three Years (Package-X)												
18	Biramitrapur Rourkela	Odisha Odisha	Improvement of Water Supply to provide safe & clean drinking water	32592 409205	32592 409205	25.64 59.89	25.64 59.89	Commissioned Ongoing	2 Years	NIL Not worked out separately	10.73 Cr.	55.52 680.43	5% 14%	100 100	3292.22 2521.23

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			conforming to drink from tap quality to Biramitrapur on Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years												

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			(Package-X)												
			Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap quality to Rourkela on Engineering, Procurem						2 Years		103.17 Cr.				

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			ent & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years (Package-IX)												
19	Sundargarh	Odisha	Improvement of Water Supply to provide safe &	56424	56424	23.65	23.65	Ongoing	2 Years	Not worked out separately	46.49 Cr.	157.86	8%	100	8239.40

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			clean drinking water conforming to drink from tap quality to Sundargarh on Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenance for a												

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Period of Three Years (Package-X)												
20	Karanjia	Odisha	Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap quality to Karanjia on	24686	24686	18.94	18.94	Ongoing	2 Years	Not worked out separately	68.46 Cr.	53.45	12%	100	27732.32

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years (Package-VI)												
21	Baripada	Odisha	Improvement of Water	130168	130168	29.65	29.65	Ongoing	2 Years	Not worked out	110.12 Cr.	264.41	7%	100	8459.84

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Supply to provide safe & clean drinking water conforming to drink from tap quality to Baripada on Engineering, Procurement & Construction (EPC) Contract Including Operation							separately					

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			& Maintenance for a Period of Three Years (Package-VIII)												
22	Rairangpur	Odisha	Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap	30577	30577	14.38	14.38	Commissioned	2 Years	Not worked out separately	59.61 Cr.	74.46	8%	100	19495.05

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			quality to Rairangpur on Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years (Package-VII)												

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
23	Udala	Odisha	Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap quality to Udala on Engineering, Procurement & Construction (EPC) Contract	14734	14734	7.89	7.89	Ongoing	2 Years	Not worked out separately	34.79 Cr.	50.43	11%	100	23612.05

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Including Operation & Maintenance for a Period of Three Years (Package-VI)												
24	Keonjhar	Odisha	Improvement of Water Supply to provide safe & clean drinking water conformin	74457	74457	21.93	21.93	Ongoing	2 Years	Not worked out separately	112.48 Cr.	148.73	12%	100	15106.71

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			g to drink from tap quality to Keonjhar on Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years												

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			(Package-II)												
25	Anandapur	Odisha	Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap quality to Anandapur on Engineering, Procurem	47012	47012	72.872	72.872	Ongoing	2 Years	Not worked out separately	103.19 Cr.	89.29	9%	100	21949.71

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			ent & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years (Package-III)												
26	Joda	Odisha	Improvement of Water Supply to provide safe &	59096	59096	26.40	26.40	Ongoing	2 Years	Not worked out separately	108.55 Cr.	100.17	12%	100	18368.42

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			clean drinking water conforming to drink from tap quality to Joda on Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenance for a Period of												

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Three Years (Package-IV)												
27	Barbil	Odisha	Improvement of Water Supply to provide safe & clean drinking water conforming to drink from tap quality to Barbil on Engineering,	88463	88463	45.89	45.89	Ongoing	2 Years	Not worked out separately	128.53 Cr.	122.07	14%	100	14529.24

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			Procurement & Construction (EPC) Contract Including Operation & Maintenance for a Period of Three Years (Package-V)												
28	Champua	Odisha	Improvement of Water Supply to provide	13671	13671	12.59	12.59	Commissioned	1 Year	NIL	32.96 Cr.	59.00	15%	100	24109.43

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			safe & clean drinking water conforming to drink from tap quality to Champuaon Engineering, Procurement & Construction (EPC) Contract Including Operation & Maintenan												

S. No.	Name of the city	Name of the state	Name of the water supply project	Total population	Population of the project area	Total Area	Area coverage under 24×7 water supply	Status of the project (Commissioned/Ongoing/ DPR approved)	Project implementation period	Cost of augmentation of source (Rs.)	Total Project cost (Rs.)	Total length of pipes	% of replacement of pipes	% of replacement of HSC	Per capita cost (Rs.)
			ce for a Period of Three Years (Package-IV)												
29	Sambalpur	Odisha	24×7 & DFT to different areas of Samblpur Municipality.	398797	398797	299.273	299.273	Ongoing	2 Years	NIL	256.71 Cr.	701.92	14%	100	6437.11

Annexure 2.2: Procedure for Consumer Survey

The process of conducting GIS based consumer survey has been illustrated by taking a case study of the survey conducted in Shimla City. To know about consumers and their locations in the city, the entire city is divided into small elements, say 500 m × 500 m. This division of the city into several elements facilitates an easy way of gathering information of the consumers. To create such elements, a 'Fishnet' (feature in GIS software) is created across the wards of the city.

Figure 1: Fishnet across the wards of the city

Creation of Fishnet: This feature is available in GIS software. Fishnet is created across all wards, as shown in Figure 1. Values of cell width and height (say, 500) for each element of the fishnet are provided.

During the creation of Fishnet, GIS software automatically labels each cell. Thus, the spread of a total of 480 elements of fishnet covers all the building boundaries of the entire city. All consumers in each element can now be easily mapped. The location of property of each consumer residing in each cell is known, and therefore, it is easier for survey teams to visit and collect information.

Mapping Consumers: All the 'Building boundaries' (in polygon shape) of the city are digitised, and its shapefile shall be created. A unique number (building_id) is assigned to each building so that it is identified and easily processed in the database. Using GIS software, the polygons are converted into the points (Figure 2). Each such point represents a consumer or their group called as a society. In the city, there is more than one consumer residing in most of the buildings, mostly the shops on the ground floor and the residential flats on the upper floors. Hence, a consumer survey should be conducted with each household (family). A consumer is identified by the 'consumer_id.'

The above fields are added in the consumer form. Survey teams should carry the forms which they have to fill by visiting the location of each consumer. The non-spatial data in the form of an Excel sheet is then joined with the common primary number of 'building_id' to the layer of the spatial shapefile of buildings which is in the form of a point. Thus, the shapefile of the consumer dataset called "consumer.shp", is created.

Figure 2: Each building is shown as red point

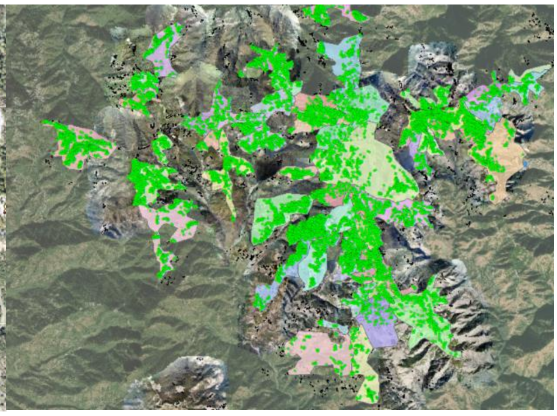


Figure 3: Consumers in each OZ of the city

Mapping consumers in each OZ: OZ is a jurisdiction of serving water by each service tank. The boundary of each OZ is created along with its shapefile called 'operational_zone.shp'. The concept of the OZ is further explained in the Chapter of the distribution system. Though the dataset of the consumer in the form of 'consumer.shp' is available, a consumer is not identified by the label of an OZ in which he resides.

In the shapefile of operational_zone.shp, there is one field called "Name" which is the name of the OZ. The task is to attach the name of the OZ to each consumer so that consumers residing in each OZ would be known.

Open GIS software with the two shape files - consumer.shp and operational_zone.shp. Choose the layer of consumers and join it with 'Join data from another location on spatial location'. Choose the layer of operational_zone to join this layer. With this, a new shapefile of consumers with the name of the OZ is created.

The connections in each OZ are shown in Figure 3. The number of consumers per OZ decides the number of DMA in each OZ.

Annexure 2.3: Identifying Existing Pipelines

Following methods are discussed here:

(1) Manual Digging Pit

Basic piece of information of underground pipelines in the city area should be obtained from the collected maps and relative background information. For identifying the existing pipelines, the following steps should be taken:

1. Make a visit to ULBs' office.
2. Compile an inventory of available maps of existing pipelines.
3. Obtain and convert available maps to digital GIS data layers - with geo referencing.
4. Take a team of consisting of a valve man, a meter reader, and an engineer from ULB to visit site along the alignment of pipelines.
5. Using Global Positioning System (GPS) device, validate alignment of pipeline after inputs from local people/staff of ULB/as-built drawings. Adequate trial pits shall be taken to confirm diameter, material of pipes.

(2) The Acoustic Detection Method

The acoustic detection method is mainly used to detect metal pipes and PVC pipes. This method depends on the attenuation speed of sound waves in different medium. The higher the attenuation rate, the softer the medium.

Principle: Acoustic detection is used to determine the connectivity and location of the pipelines. In this method, an acoustic signal is applied at one end of the pipe in the form of vibrations and these vibrations are detected using a receiver. A detection of same frequency of the sound by the receiver at surface will indicate the connectivity and layout of the pipe. Taking into consideration the attenuation of the sound waves propagating through the medium, one may be able to detect any faults in the connection between the pipes. The experimental process is shown in Figure 1.

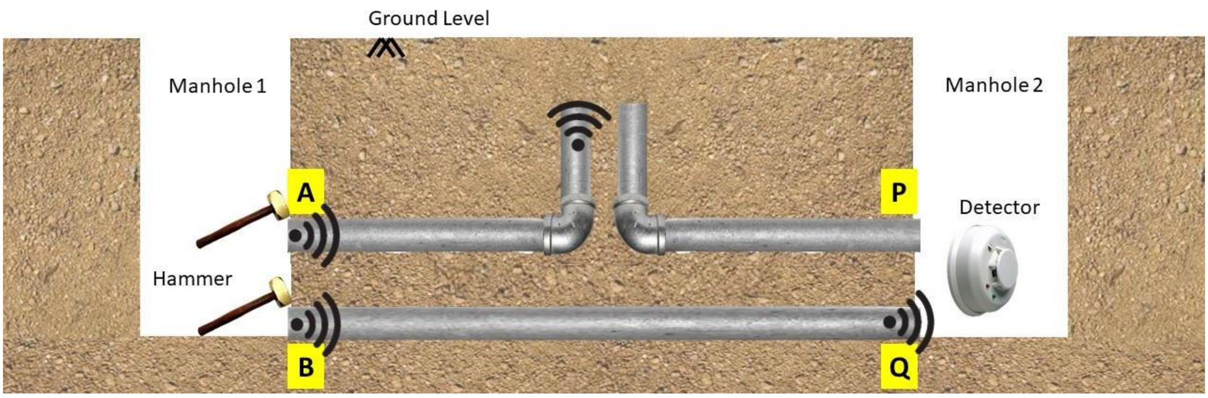


Figure 1: Hammer and Stethoscope method for pipeline detection

Field execution procedure is as below:

- Identify the wells/manhole to access the section of pipelines.
- Consider two manholes. Mark all the pipes if multiple pipelines are available in the manhole.

- The hammer is banged on any of the pipes in the first manhole. The frequency of the signal transmitted in the pipe is noted using a stethoscope.
- A stethoscope is used to detect the vibrations of the pipes in the second manhole. If any vibration is detected, the frequency is compared to the frequency of signal at the banging end.
- If the frequency matches, it indicates a direct connection of these pipes.
- In this case, when hammer is struck at ‘Pipe B’ in manhole 1 (Figure 2.4), the detector detects a signal at ‘Pipe Q’ in manhole 2. This indicates a direct connection of Pipe B and Pipe Q. Thus, BQ is a single pipe.

Shortcomings: This method is reliable in limited ranges depending upon the material of the pipes. In long distances, the signal attenuates to zero hence, no signal is detected in the second manhole.

Pipe fittings cause scattering of the signal and hence an attenuation of the signal. A pipe stretch having many fittings can attenuate the signal to zero and hence, the connection may not be detected.

The method using hammers to pass a signal through the pipe material does not give the direction or the layout of the pipelines.

Acoustic detection does not give any information about the depth and other dimensions of the pipeline.

(3) Electromagnetic Detection

This method is suitable for metallic pipelines.

Principle: This is a widely used method for detection. It uses the principle of electromagnetic induction for determining underground utilities. The utilities can detect pipes either by an active signal, or a passive signal. Active signal is to be used for the detection in case of pipelines as they do not carry any power through them. An active signal is generated by a transmitter and applied to a pipeline which is then detected by a receiver. For detecting pipelines, one of the following methods of transmission may be used:

(i) Direct connection: In this method, the transmitter is directly connected to the pipe at the point of access, and a signal of particular frequency are passed to the pipe (Figure 2). This signal is then detected using a receiver.

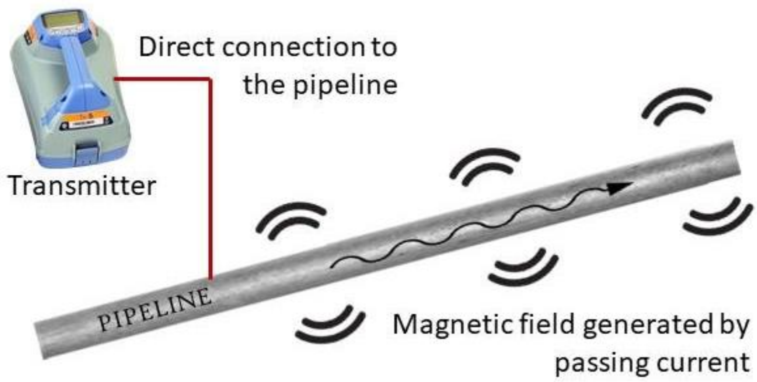


Figure 2: Direct connection method for electromagnetic detection

(ii) Induction: In induction, the transmitter passes a current through a coil (Figure 3) which produces a magnetic field. The magnetic field generates a current in any metallic pipe cutting through this magnetic field. This current can be detected using a receiver.

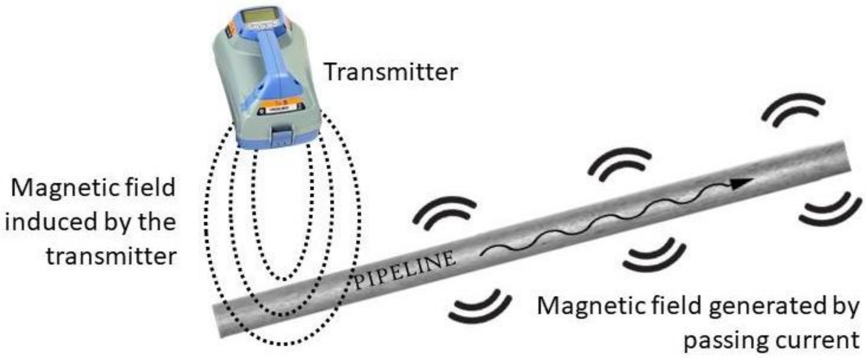


Figure 3: Induction method for electromagnetic detection

(iii) Clamping: This method provides results similar to direct method but without direct electrical contact. A toroidal wire is clamped around the pipeline (Figure 4) at access point. On passing the current through the wire, it is magnetised and induces a strong current in the pipeline.

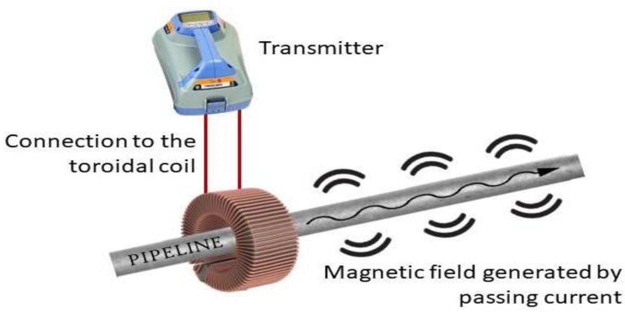


Figure 4: Clamping method for electromagnetic detection

(iv) Remote transmitter: Sondes are battery-operated compact transmitters which may be mounted on flexible rods and pushed inside the plastic and other non-metallic pipelines. These transmitters (Figure 5) can then be detected from surface by any receiver equipped with an antenna to detect electric and magnetic fields.

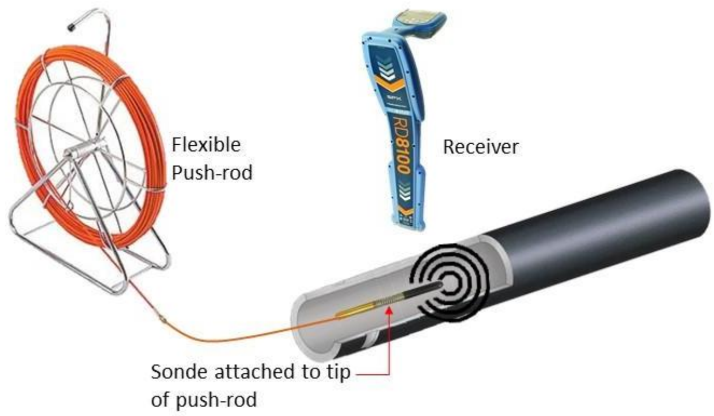


Figure 5: Using remote transmitter for electromagnetic detection.

For using the above device field, execution procedure is explained in Figure 6.

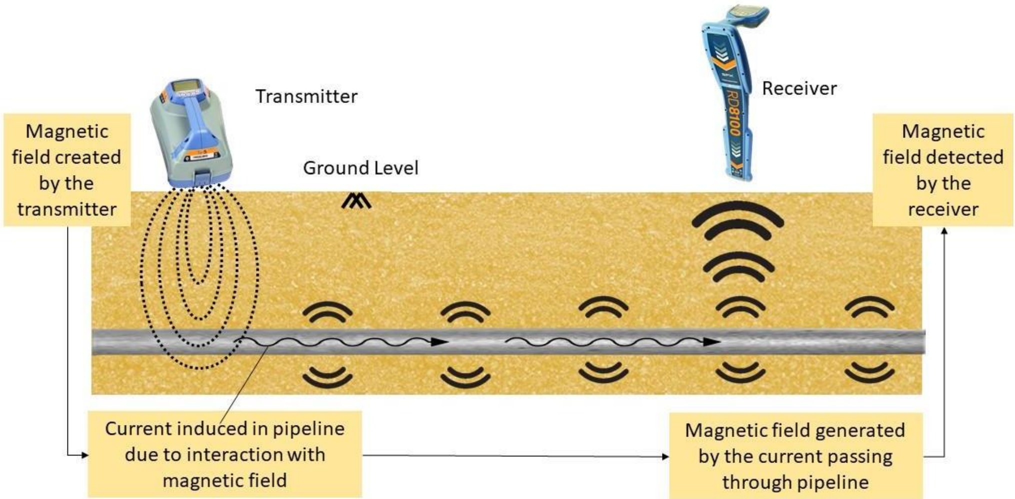


Figure 6: Field procedure for pipeline detection using electromagnetic method

Individual pipe detection:

i. Collect existing maps, drawings, layout information available for the network in study.
ii. Locate the start points, endpoints, pumping stations, nodes and other important points in the pipe network from the available data or by field survey.
iii. One of the above four methods is chosen based on the access to the pipelines.
iv. It is preferable to use the direct current method or the clamping method for individual pipe detection as other pipelines do not catch the signal.
v. The signal is applied to the pipe using the chosen method.
vi. The receiver is swayed side to side over the ground to detect the signal from the pipeline.
vii. The location of pipeline is marked on detection and receiver is moved forward slowly, swaying side to side.
viii. This way the entire path of the pipeline is traced.
ix. This data can be collected and may be directly marked on the ground or plotted in a GIS map of the area.

(v) Mapping the pipe network:

In this method, initially an inductive transmitter is used. The entire area is covered by sweeping the area in a grid pattern (Figure 7).

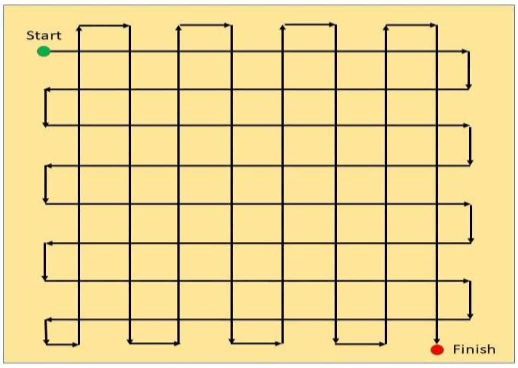


Figure 7: Path to be followed for mapping pipelines

a) The transmitter is placed at the initial position.
b) The receiver is swept along lines in a grid pattern along both perpendicular directions.
c) When a signal is received, the transmitter is shifted to that position and the signal is confirmed with the receiver. A confirmed signal is noted down.
d) The transmitter is re-positioned at the initial position and the sweeping is resumed.
e) Once the entire grid pattern is swept, the individual pipes are traced by using the direct current method or the clamping method based on the signals received from the grid sweeping.
f) The location of the pipes obtained are then plotted suitably or fed into a GIS map of the area.

Shortcomings of electromagnetic induction method:

a) Non-metallic and high carbon pipes cannot be detected using this method as they do not conduct electricity. The method may be used by inserting energised lines into such pipes or by pigging methods, but such techniques are very sophisticated and have their own limitations.
b) While using the induction method, other metallic objects may catch the signal and can be detected by the receiver.

(4) Ground Penetrating Radar (GPR)

This method is suitable for both metallic and non-metallic pipelines. GPR is commonly used as a non-destructive testing (NDT) technique that allows, among others, the detection and localisation of buried utilities without any damage to the surface.

Principle: This technology is especially useful for the detection of non-metallic utility lines which were earlier considered to be non-locatable. This method uses emission, reflection, and detection of radio waves for locating underground utilities. The underground utility lines reflect the radio waves; these waves are detected by the receiver. Detection is indicated by a hyperbolic mark in the profile generated for the cross section being examined.

Field execution procedure is shown in Figure 8. For individual pipe detection following steps are undertaken:

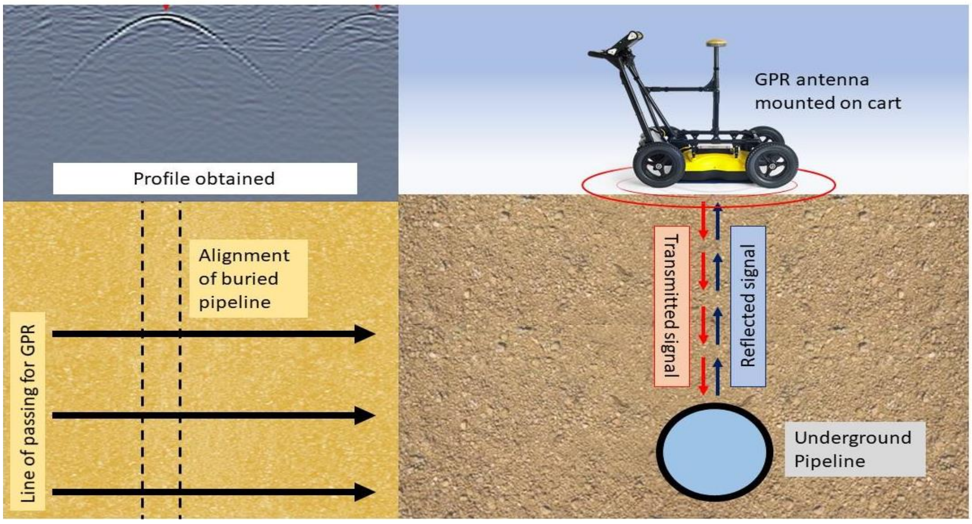


Figure: 8: GPR method for pipeline detection

i. Collect existing maps, drawings, layout information available for the network in study.
ii. Locate the start points, endpoints, pumping stations, nodes, and other important points in the pipe network from the available data or by field survey.
iii. The start point of the pipeline is identified from where the survey is to be started.
iv. The GPR mounted over a cart is swept along parallel line perpendicular to the orientation of the pipeline.
v. This gives a profile of the ground for that particular section along the line of sweep.
vi. A hyperbolic mark in this profile will indicate presence of an underground object. The location of the mark is marked on the ground for each section.
vii. The location of the pipeline is marked on each line of sweep. These points can be connected to obtain the layout of the pipe inside the ground.

Mapping the pipe network is carried out as below:

i. For mapping the entire pipe network, the entire area is divided into a grid.
ii. The GPR is swept along each of the parallel lines in both the perpendicular directions.
iii. It is preferable to conduct all the sweeps in the same direction.
iv. The device collects the GPS location of sweep and length of sweep (Figure 9) from the wheel rotation.
v. The GPR equipment equipped with appropriate software can automatically analyse all the cross-sections data and generate a map of all the utilities in that area. This can be connected to the GPS data collected for each sweep and plotted on a GIS platform.

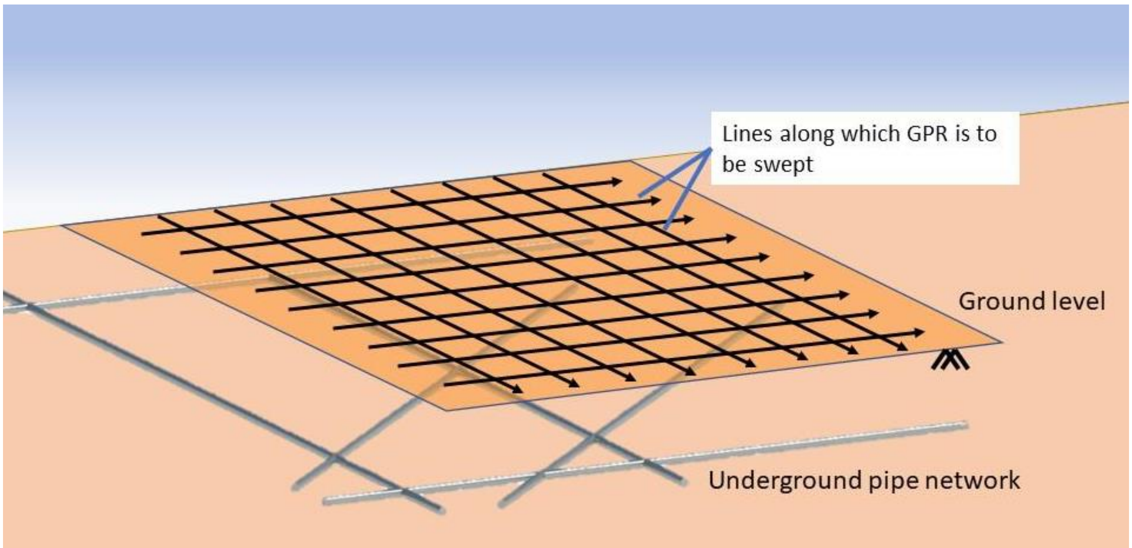


Figure 9: GPR method for pipeline detection

Shortcomings of GPR detection: The data can be difficult to interpret and requires careful observation of the irregularities to detect any underground object. The equipment used can be affected by the interference from other devices working with radio wave frequencies. The radio waves may also be absorbed by certain soils like clays and saline soil.

Annexure 2.4: Condition Assessment

Condition Assessment of Water Pipelines and Appurtenances

Condition assessment may be defined as identifying the likelihood that an asset will continue to perform its required function. As part of condition assessment, data and information are collected through direct and indirect methods. Data is then analysed to determine the physical characteristics of the pipe and how they may impact the pipeline's likelihood that it will leak, break, or otherwise fail to perform.

There is rapid urbanising in India. Huge capital investments have been allocated to replaces the existing water pipeline networks. Most of these projects are designed to be PPPs; one of major tasks is to understand the percentage of the network needing replacement. The DPRs makes it compulsory for the private players to quantify the replacement. DPR comprises of tender notice, Bill of quantities, specifications, instructions to contractor, etc. Any water supply scheme needs preparation of tender which requires DPR and which in turn requires quantities of pipes (existing and newly proposed). In order to find out quantities of existing pipes to be replaced.

As per the estimate, the pipeline replacement accounts a major amount of the project cost. However, without having a proper condition assessment technology and methodology, the estimation of the condition of these networks is often wrong. If the assumption of the pipeline replacement is wrong, it leads to major challenges such as:

Delayed understanding of the actual replacement leads to significant challenges further like overrun of budgets, delaying of projects, etc.

- The ULB/private operator will not be able to bring down the level of NRW at 15%.
- If the project is completed without the right amount of pipeline replacement, it will lead to regular leakages in the water pipelines and blockages in the sewer lines.

Thus, the assessment of the pipelines before finalising the DPR should be a 'must' before finalising the final value of the projects. Condition assessment of the pipeline shall be done periodically.

Advantages of Condition Assessment

Following are the advantages:

1) Helps classify the pipeline network under three parameters (as a percentage of the total network):
   a. pipeline to be replaced;
   b. pipeline to be rehabilitated;
   c. pipelined to be retained as is.
2) Estimate the likelihood that a water mains/distribution line may continue to provide satisfactory service, both now and in the future.
3) Helps determine the remaining service life of the water mains and distribution lines.
4) Make better decisions regarding maintenance of the pipelines by allowing some water mains/distribution lines to remain in service longer; prevent some pipeline

failures from occurring by intervening sooner; make decisions more confidently (with less chance of error).

5) Find active leaks.

Condition assessment of pipes and appurtenances have to be mapped on GIS asset management maps with present condition of pipes, history of burst leaks and material of composition, leak locations and severities, and suitable financial plans in place for every city which is going for conversion to 24×7 water supply system.

Methods of Condition Assessment

Three methods condition assessment are as follows:

1) Robotic pipeline inspection
2) Inline Tethered Pipeline Inspection
3) External Non-Destructive Test (NDT) techniques

(1) Robotic Pipeline Inspection

Many cities are using new technologies to study condition assessment of their pipelines. A robotic equipment is inserted into the pipeline (Figure 1) through an opening, remotely controlled with the help of a tether. The live video feed from the camera mounted on robot is obtained at the base station. The device is then navigated inside the pipeline using visual feedback at the base station. This robotic equipment technology is used for internal pipeline assessment and mapping.

Figure 1: Glimpses of the site preparation for launching the inspection equipment.

(Source: Tamil Nadu Water Investment Company Ltd.)

Condition assessment: After assessment of various technologies, crawler-based robotic pipeline inspection systems capable of inspecting pipelines as small as 90 mm in diameter using CCTV are proposed, which provides a deterministic assessment of the system based on the images, videos and data analytics derived from the video images. It generally needs three steps: Pre-inspection, inspection, and post-inspection.

During pre-inspection, the pipelines are identified and a sampling percentage (5%-10%) is decided according to the size of the network. Furthermore, according to various factors like topography, the density of population, history of leaks, pipeline material, and age, the sampling lengths are distributed over the entire network.

During the inspection, crawler robots (Figure 2) with capability to inspect pipelines as small as 90 mm diameter are used that collect visual data as well as other data with the help of an array of sensors which include laser profiler, encoder, etc.

Figure 2: Crawler robots for various pipeline sizes

Post inspection, the analysis is done to grade the severity of the network as per WRC guidelines. Additionally, data from laser profiling is analysed to find out the effective internal diameter of the pipeline, its ovality.

Cloud-Based Data Management for Pipeline: Data digitisation and visualisation provide data and information that would help to prioritise decision-making.

A pipeline management system for pipelines should have the following:

1) a management system for tracking the various defects in the pipelines, their rehabilitation status and success rate;
2) a visualisation system that records the kind of defects tagged with GIS data along with pipeline parameters, 24/7, right in the cloud. Utility managers can watch the recorded video, add observations, and manage quality;
3) an automatic report generation system filtered based on location, dates, etc., for decentralised decision-making and to strengthen the case for correct DPRs.

Prediction algorithm and remaining life of the pipeline:

Cities can benefit greatly by leveraging the data of their condition assessment, corrosion, and scaling to identify the pipeline risks. It helps to assess the remaining life of the pipeline based on machine learning and pipeline degradation and evaluate future safe operating strategies, including re-inspection and appropriate maintenance schedules. As a result, it can minimise the possibility of pipeline failures.

Application of robotic inspection can be made for existing water pipeline network as follows:

(a) New Water Pipeline Network

1) GIS mapping of water pipeline assets should be done at the time of new water pipeline laying works.
2) Conditional Assessment of newly laid water pipelines from 75 mm and above should be done for the newly laid pipelines so as to create a milestone data for future reference and also to ensure the quality of construction. Such an assessment should necessarily include checking of welding joints, compression, rupture, visible cracks, capital silt and debris in the line.
3) The use of Crawler-based motorized and remotely-controlled Robotic pipeline inspection systems (Figure 3) with Pan-Tilt-Zoom (PTZ) cameras can be used. It can be used for pipe diameters of 150 mm and above.
4) The use of remotely controlled and motorized inspection system with the ability to negotiate 90° horizontal and vertical bends in pipelines of diameter 100 mm and above.
5) LASER Profiling can be used for checking for any deformation or compression in Large Diameter Pipelines like Water Mains and cross-country water pipelines.

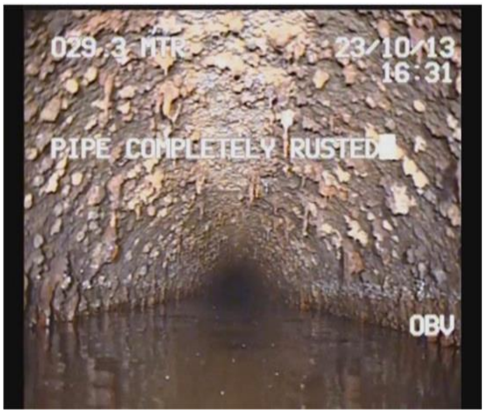


Figure 3: Robotic Water Pipeline Conditional Assessment

(b) Existing Water Pipeline Network

1) It is imperative to check the existing network for defects on a timely basis to resolve issues with the pipelines and also to maintain a pre-emptive check on future issues.
2) For checking of existing water pipeline assets in non-pressurised condition, crawler-based motorised and remotely controlled robotic pipeline inspection systems may be used for checking the pipeline for defects. These can work in water pipelines without the requirement of removing/pumping out water.
3) For the checking of pressurised water pipeline assets, Hydro Chute-based tethered inspection systems having on-board camera and acoustic sensor for detection of leaks can also be used.
4) In case it is not possible to keep the line pressurised during inspection of water mains, it is advised to use crawler-based motorised and remotely controlled robotic pipeline inspection systems with long distance inspection capability of 1 km in single running length. The system will be able to inspect up to 1 km of pipe length in either direction of the point of insertion. These inspection systems can be used without incurring the cost of removal of water. Such a system is to be used for inspection of water mains and cross-country water pipelines.
5) Simultaneous inspection and condition assessment of pipelines shall organically accumulate milestone data for future analysis. It is advised to map the lines under rectification/maintenance on the central GIS system.
6) In order to establish GIS map of the existing water pipeline assets, GPR survey of existing network should be performed. Areas where there is doubt regarding the pipelines due to overlap with other existing underground pipeline networks (i.e., electrical, Internet, sewer, etc.), an intrusive inspection using Robotic Inspection ROVs (Remotely Operated Vehicles) should be performed to confirm the location of the pipe network. In densely populated cities with a large underground pipelines infrastructure of various utilities, GPR survey may not provide the desired results. In such a case, intrusive inspections should be used for mapping of the water pipeline network.

(2) Inline Tethered Pipeline Inspection

There are some platforms that can make inspection of pressurised water pipelines of diameters 150 mm or more. It can be done without disrupting regular service. The device is inserted into running pipeline through access point having size of 50 mm. The inserted device collects the status of pipeline information up to 1 to 1.6 km in one operation. The suspected leaks and other visual irregularities can be seen by the operator using the tethered device. The location of irregular anomalies can be marked real-time.

(3) External NDT techniques

(a) Radiography: One of the common methods of NDT is radiography. A radiographic film is placed behind pipeline and using the source of the radiation, the anomaly can be recorded.

(b) Ultrasonic: This method of NDT is useful to check integrity of welded pipeline. High frequency acoustic waves penetrates the pipeline. The wave is attenuated, the change in pipe material's density is noted. The return signal indicates the pipeline defects.

(c) Other methods: Eddy current is used in which a time varying magnetic field is induced in the pipeline. The change in impedance of the magnetic coil as it traverses the pipeline is used to identify different characteristics.

Annexure 2.5: Drinking Water Quality

Drinking water shall comply with the requirements given in Tables 1 to 4 which are reproduced from IS 10500:2012. The analysis of pesticide residues given in Table 3 shall be conducted by a recognised laboratory using internationally established test method meeting the residue limits as given in Table 5.

Table 1 Organoleptic and Physical Parameters
(*Foreword* and *Clause* 4)

Sl No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to Part of IS 3025	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
i)	Colour, Hazen units, *Max*	5	15	Part 4	Extended to 15 only, if toxic substances are not suspected in absence of alternate sources
ii)	Odour	Agreeable	Agreeable	Part 5	a) Test cold and when heated b) Test at several dilutions
iii)	*p*H value	6.5-8.5	No relaxation	Part 11	—
iv)	Taste	Agreeable	Agreeable	Parts 7 and 8	Test to be conducted only after safety has been established
v)	Turbidity, NTU, *Max*	1	5	Part 10	—
vi)	Total dissolved solids, mg/l, *Max*	500	2 000	Part 16	—

NOTE — It is recommended that the acceptable limit is to be implemented. Values in excess of those mentioned under ‘acceptable’ render the water not suitable, but still may be tolerated in the absence of an alternative source but up to the limits indicated under ‘permissible limit in the absence of alternate source’ in col 4, above which the sources will have to be rejected.

Table 2 General Parameters Concerning Substances Undesirable in Excessive Amounts
(*Foreword* and *Clause* 4)

Sl No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
i)	Aluminium (as Al), mg/l, *Max*	0.03	0.2	IS 3025 (Part 55)	—
ii)	Ammonia (as total ammonia-N), mg/l, *Max*	0.5	No relaxation	IS 3025 (Part 34)	—
iii)	Anionic detergents (as MBAS) mg/l, *Max*	0.2	1.0	Annex K of IS 13428	—
iv)	Barium (as Ba), mg/l, *Max*	0.7	No relaxation	Annex F of IS 13428* or IS 15302	—
v)	Boron (as B), mg/l, *Max*	0.5	1.0	IS 3025 (Part 57)	—
vi)	Calcium (as Ca), mg/l, *Max*	75	200	IS 3025 (Part 40)	—
vii)	Chloramines (as Cl_2), mg/l, *Max*	4.0	No relaxation	IS 3025 (Part 26)* or APHA 4500-Cl G	—
viii)	Chloride (as Cl), mg/l, *Max*	250	1 000	IS 3025 (Part 32)	—
ix)	Copper (as Cu), mg/l, *Max*	0.05	1.5	IS 3025 (Part 42)	—
x)	Fluoride (as F) mg/l, *Max*	1.0	1.5	IS 3025 (Part 60)	—
xi)	Free residual chlorine, mg/l, *Min*	0.2	1	IS 3025 (Part 26)	To be applicable only when water is chlorinated. Tested at consumer end. When protection against viral infection is required, it should be minimum 0.5 mg/l
xii)	Iron (as Fe), mg/l, *Max*	0.3	No relaxation	IS 3025 (Part 53)	Total concentration of manganese (as Mn) and iron (as Fe) shall not exceed 0.3 mg/l
xiii)	Magnesium (as Mg), mg/l, *Max*	30	100	IS 3025 (Part 46)	—
xiv)	Manganese (as Mn), mg/l, *Max*	0.1	0.3	IS 3025 (Part 59)	Total concentration of manganese (as Mn) and iron (as Fe) shall not exceed 0.3 mg/l
xv)	Mineral oil, mg/l, *Max*	0.5	No relaxation	Clause **6** of IS 3025 (Part 39) Infrared partition method	—
xvi)	Nitrate (as NO_3), mg/l, *Max*	45	No relaxation	IS 3025 (Part 34)	—
xvii)	Phenolic compounds (as C_6H_5OH), mg/l, *Max*	0.001	0.002	IS 3025 (Part 43)	—
xviii)	Selenium (as Se), mg/l, *Max*	0.01	No relaxation	IS 3025 (Part 56) or IS 15303*	—
xix)	Silver (as Ag), mg/l, *Max*	0.1	No relaxation	Annex J of IS 13428	—
xx)	Sulphate (as SO_4) mg/l, *Max*	200	400	IS 3025 (Part 24)	May be extended to 400 provided that Magnesium does not exceed 30
xxi)	Sulphide (as H_2S), mg/l, *Max*	0.05	No relaxation	IS 3025 (Part 29)	—
xxii)	Total alkalinity as calcium carbonate, mg/l, *Max*	200	600	IS 3025 (Part 23)	—
xxiii)	Total hardness (as $CaCO_3$), mg/l, *Max*	200	600	IS 3025 (Part 21)	—
xxiv)	Zinc (as Zn), mg/l, *Max*	5	15	IS 3025 (Part 49)	—

NOTES

1 In case of dispute, the method indicated by '*' shall be the referee method.

2 It is recommended that the acceptable limit is to be implemented. Values in excess of those mentioned under 'acceptable' render the water not suitable, but still may be tolerated in the absence of an alternative source but up to the limits indicated under 'permissible limit in the absence of alternate source' in col 4, above which the sources will have to be rejected.

Table 3 Parameters Concerning Toxic Substances
(*Foreword* and *Clause* 4)

Sl No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
i)	Cadmium (as Cd), mg/l, *Max*	0.003	No relaxation	IS 3025 (Part 41)	—
ii)	Cyanide (as CN), mg/l, *Max*	0.05	No relaxation	IS 3025 (Part 27)	—
iii)	Lead (as Pb), mg/l, *Max*	0.01	No relaxation	IS 3025 (Part 47)	—
iv)	Mercury (as Hg), mg/l, *Max*	0.001	No relaxation	IS 3025 (Part 48)/ Mercury analyser	—
v)	Molybdenum (as Mo), mg/l, *Max*	0.07	No relaxation	IS 3025 (Part 2)	—
vi)	Nickel (as Ni), mg/l, *Max*	0.02	No relaxation	IS 3025 (Part 54)	—
vii)	Pesticides, µg/l, *Max*	See Table 5	No relaxation	See Table 5	—
viii)	Polychlorinated biphenyls, mg/l, *Max*	0.000 5	No relaxation	ASTM 5175*	— or APHA 6630
ix)	Polynuclear aromatic hydro-carbons (as PAH), mg/l, *Max*	0.000 1	No relaxation	APHA 6440	—
x)	Total arsenic (as As), mg/l, *Max*	0.01	0.05	IS 3025 (Part 37)	—
xi)	Total chromium (as Cr), mg/l, *Max*	0.05	No relaxation	IS 3025 (Part 52)	—
xii)	Trihalomethanes:				
	a) Bromoform, mg/l, *Max*	0.1	No relaxation	ASTM D 3973-85* or APHA 6232	—
	b) Dibromochloromethane, mg/l, *Max*	0.1	No relaxation	ASTM D 3973-85* or APHA 6232	—
	c) Bromodichloromethane, mg/l, *Max*	0.06	No relaxation	ASTM D 3973-85* or APHA 6232	—
	d) Chloroform, mg/l, *Max*	0.2	No relaxation	ASTM D 3973-85* or APHA 6232	—

NOTES

1 In case of dispute, the method indicated by '*' shall be the referee method.

2 It is recommended that the acceptable limit is to be implemented. Values in excess of those mentioned under 'acceptable' render the water not suitable, but still may be tolerated in the absence of an alternative source but up to the limits indicated under 'permissible limit in the absence of alternate source' in col 4, above which the sources will have to be rejected.

Table 4 Parameters Concerning Radioactive Substances
(*Foreword* and *Clause* 4)

Sl No.	Characteristic	Requirement (Acceptable Limit)	Permissible Limit in the Absence of Alternate Source	Method of Test, Ref to Part of IS 14194	Remarks
(1)	(2)	(3)	(4)	(5)	(6)
i)	Radioactive materials:				
	a) Alpha emitters Bq/l, *Max*	0.1	No relaxation	Part 2	—
	b) Beta emitters Bq/l, *Max*	1.0	No relaxation	Part 1	—

NOTE — It is recommended that the acceptable limit is to be implemented. Values in excess of those mentioned under 'acceptable' render the water not suitable, but still may be tolerated in the absence of an alternative source but up to the limits indicated under 'permissible limit in the absence of alternate source' in col 4, above which the sources will have to be rejected.

Table 5 Pesticide Residues Limits and Test Method
(*Foreword* and *Table* 3)

Sl No.	Pesticide	Limit µg/l	Method of Test, Ref to	
			USEPA	AOAC/ ISO
(1)	(2)	(3)	(4)	(5)
i)	Alachlor	20	525.2, 507	—
ii)	Atrazine	2	525.2, 8141 A	—
iii)	Aldrin/ Dieldrin	0.03	508	—
iv)	Alpha HCH	0.01	508	—
v)	Beta HCH	0.04	508	—
vi)	Butachlor	125	525.2, 8141 A	—
vii)	Chlorpyriphos	30	525.2, 8141 A	—
viii)	Delta HCH	0.04	508	—
ix)	2,4- Dichlorophenoxyacetic acid	30	515.1	—
x)	DDT (*o, p* and *p, p* – Isomers of DDT, DDE and DDD)	1	508	AOAC 990.06
xi)	Endosulfan (alpha, beta, and sulphate)	0.4	508	AOAC 990.06
xii)	Ethion	3	1657 A	—
xiii)	Gamma — HCH (Lindane)	2	508	AOAC 990.06
xiv)	Isoproturon	9	532	—
xv)	Malathion	190	8141 A	—
xvi)	Methyl parathion	0.3	8141 A	ISO 10695
xvii)	Monocrotophos	1	8141 A	—
xviii)	Phorate	2	8141 A	—

NOTE — Test methods are for guidance and reference for testing laboratory. In case of two methods, USEPA method shall be the reference method.

Table 6 Bacteriological Quality of Drinking Water[1)]
(*Clause* 4.1.1)

Sl No.	Organisms	Requirements
(1)	(2)	(3)
i)	*All water intended for drinking*:	
	a) *E. coli* or thermotolerant coliform bacteria[2), 3)]	Shall not be detectable in any 100 ml sample
ii)	*Treated water entering the distribution system*:	
	a) *E. coli* or thermotolerant coliform bacteria[2)]	Shall not be detectable in any 100 ml sample
	b) Total coliform bacteria	Shall not be detectable in any 100 ml sample
iii)	*Treated water in the distribution system*:	
	a) *E. coli* or thermotolerant coliform bacteria	Shall not be detectable in any 100 ml sample
	b) Total coliform bacteria	Shall not be detectable in any 100 ml sample

[1)] Immediate investigative action shall be taken if either *E.coli* or total coliform bacteria are detected. The minimum action in the case of total coliform bacteria is repeat sampling; if these bacteria are detected in the repeat sample, the cause shall be determined by immediate further investigation.

[2)] Although, *E. coli* is the more precise indicator of faecal pollution, the count of thermotolerant coliform bacteria is an acceptable alternative. If necessary, proper confirmatory tests shall be carried out. Total coliform bacteria are not acceptable indicators of the sanitary quality of rural water supplies, particularly in tropical areas where many bacteria of no sanitary significance occur in almost all untreated supplies.

[3)] It is recognized that, in the great majority of rural water supplies in developing countries, faecal contamination is widespread. Under these conditions, the national surveillance agency should set medium-term targets for progressive improvement of water supplies.

Annexure 2.6: Land required for Water Supply Infrastructure

City planners should earmark the land required for water supply infrastructure in the master plan of the city for the next 30 years. This task should be done in consultation with water supply engineers.

The land is required for WTPs, sumps and ESRs, etc. When the land for water supply infrastructure is not available, the city planners should allow development of water infrastructure above or below recreational amenities or parks, stadiums, etc.

City planners should be informed. This solution can solve the problem of constructing sump and pump houses required for direct pumping to distribution pipe networks. The tentative land required for new ESRs tanks is shown in Table 1.

Table 1: Land required for new Elevated Service tanks

SN	Capacity (Cum)	SWD (m)	Indicative Plot Size*	Indicative Land Required*(Sq. m)
1	50	3	14 m × 14 m	196
2	75	3	15 m × 15 m	225
3	100	3	16 m × 16 m	256
4	125	3	16 m × 16 m	256
5	150	3	17 m × 17 m	289
6	200	3	22 m × 22 m	484
7	250	3	24 m × 24 m	576
8	300	3	24 m × 24 m	576
9	350	3	25 m × 25 m	625
10	400	3	26 m × 26 m	676
11	450	3	27 m × 27 m	729
12	500	3	28 m × 28 m	784
13	750	4	33 m × 33 m	1089
14	1000	4	35 m × 35 m	1225
15	1250	4	37 m × 37 m	1369
16	1500	4	41 m × 41 m	1681
17	2000	5	42 m × 42 m	1764
18	2500	5	49 m × 49 m	2401
19	3000	5	51 m × 51 m	2601

*Note: These are bare minimum area and plot sizes. The size may be rectangular or circular depending upon the availability of the land. The land should have access road and power connection facility available.

Annexure 2.7: Ward-Wise Population by Equivalent Area Method

The method presented here aims to find a city's ward-wise future population and population density. The method has been adopted using a GIS technology. A case study of the water supply project of one coastal city is discussed here. The reason for selecting this town is that it includes practically all types of uses, including commercial, parks, playgrounds, public and semi-public utility, industrial and operational industrial areas, water bodies, etc.

The objective is to explain the steps required to prepare a GIS layer of future population density and how it is used to allocate the demands to the water distribution system nodes.

1. Land Use Maps

Land use maps of a city/town comprise the spatial information/data of the various physical land uses like that of the residential area, areas of commercial activity, transportation, parks and gardens, forest land, etc. These land use maps are generally included in the CDP. CDP map of the town is collected and is georeferenced. After the geo-referencing process, the polylines of roads, buildings, etc., are exported to form the shapefile of the different types of land, which are shown in Figure 1.

Figure 1: Land use map of town

Figure 2: Overlapping of layers of 'land use' map and 'wards'

In the analysis/design of the distribution system of water supply, ward-wise population density for the present, next 15 years, and 30 years needs to be computed.

1.1 Determining Future Population Density of Wards

There are 23 wards in the town. The polygons of different wards are digitised, and a shapefile of the boundary of all the 23 wards of the town has created the layer which is shown in Figure 3.

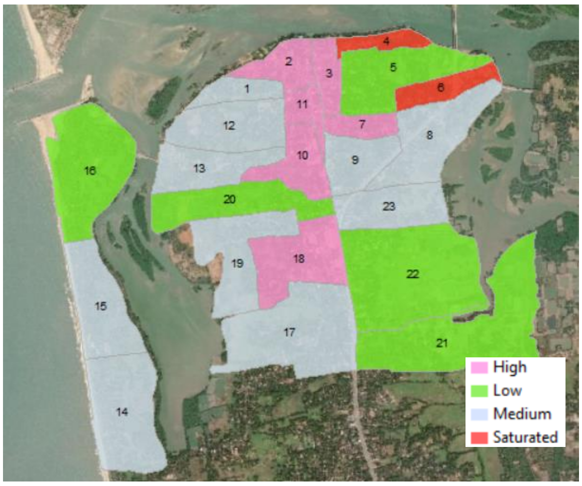


Figure 3: Wards of the city

Figure 4: Wards with density

Ward-Wise Land Use Area: Though the map showing all types of land uses for the entire town is available; it is necessary to find out different types of land use areas for each individual ward. Two overlapping shape files, (a) land use map (Figure 1) and (b) ward boundaries (Figure 3), are used to form overlapping layers in Figure 2. Different types of land use areas for each ward are formed using the GIS tool which is used to split wards into different land uses.

After splitting the wards, shape files of each ward with corresponding land use areas are obtained. Information from these shape files, after splitting the wards, is collected, which is shown in Columns 5 to 8 of Table 1.

Since the population density of each ward with respect to land use is to be found out, it is required to find out the equivalent area of each ward. While determining equivalent area, the general factors such as 100% for residential, 25% for the public, and 10% for industries and agriculture have been used, which is practised in Maharashtra Jeevan Pradhikaran (MJP). The computation of the equivalent area is shown in Table 1.

Table 1: Computation of ward-wise Equivalent area and Projected ward-wise population for design year

Ward No.	area (Ha)	Population	population density on total area 2011 (P/Ha)	Type of Land Use from GIS map: Residential (Ha)	Public* Area (Ha)	Industrial** (Ha)	No Man's Area*** (Ha)	Eq. Area (Ha)	Present Population Density on Eq. area	Density: Main Category	Sub-category	Iteration=1: Projected density for design year 2046 (p/Ha)	Projected population for design year 2046	Iteration=2: Projected density for design year 2046 (p/Ha)	Projected population for design year 2046	Iteration=3: Projected density for design year 2046 (p/Ha)	Projected population for design year 2046
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
22	97.5	1169	12	83.4	11	0.1	3.1	86.12	13.57	Low	L1#	25	2153	25	2153	25	2153
16	66.6	1151	17	66	0.6	0	0	66.18	17.39	Low	L1	25	1654	25	1654	25	1654
21	90.8	1389	15	74.8	15.9	0	0	78.79	17.63	Low	L1	25	1970	25	1970	25	1970
5	47.8	1348	28	43.6	2.8	1.5	0	44.42	30.35	Low	L2	40	1777	35	1555	35	1555
20	42.9	1163	27	36.3	6.6	0	0	37.99	30.61	Low	L2	40	1520	35	1330	35	1330
17	77.4	2092	27	63.4	13.9	0	0.2	66.85	31.29	Medium	M1	55	3677	50	3343	45	3008
14	59.7	1799	30	56.1	0.9	0	2.7	56.35	31.93	Medium	M1	55	3099	50	2818	45	2536
15	43.9	1445	33	41.2	1.8	0	1	41.63	34.71	Medium	M1	55	2290	50	2082	45	1873
12	41.7	1388	33	34.9	6.8	0	0	36.59	37.93	Medium	M1	55	2013	50	1830	45	1647

Ward No.	area (Ha)	Population	population density on total area 2011 (P/Ha)	Type of Land Use from GIS map				Eq. Area (Ha)	Present Population Density on Eq. area	Density		Iteration=1		Iteration=2		Iteration=3	
				Residential (Ha)	Public* Area (Ha)	Industrial** (Ha)	No Man's Area *** (Ha)			Main Category	Sub-category	Projected density for design year 2046 (p/Ha)	Projected population for design year 2046	Projected density for design year 2046 (p/Ha)	Projected population for design year 2046	Projected density for design year 2046 (p/Ha)	Projected population for design year 2046
19	46.1	1594	35	38.7	7.2	0	0.1	40.56	39.3	Medium	M1	55	2231	50	2028	45	1825
13	32.9	1224	37	29.6	3.3	0	0	30.44	40.21	Medium	M2	70	2131	60	1826	55	1674
8	48.3	1706	35	40.3	7.1	0.3	0.7	42.12	40.51	Medium	M2	70	2948	60	2527	60	2527
9	27	948	35	21.4	5.2	0	0.4	22.72	41.73	Medium	M2	70	1590	60	1363	60	1363
1	14.7	588	40	13.9	0.7	0.1	0	14.06	41.83	Medium	M2	70	984	60	844	60	844
23	35.1	1425	41	30.8	4.2	0	0.1	31.84	44.75	Medium	M2	70	2229	60	1911	60	1911
18	44.5	1954	44	29.5	15	0	0	33.22	58.81	High	H1	90	2990	80	2658	80	2658
10	26.8	1269	47	19.3	7.4	0	0	21.18	59.92	High	H1	90	1906	80	1694	80	1694
2	20.4	1108	54	14.4	5.5	0.2	0.4	15.75	70.33	High	H1	90	1418	80	1260	80	1260
11	12.4	552	44	6	6.4	0	0	7.63	72.37	High	H1	90	686	80	610	80	610

Ward No.	area (Ha)	Population	population density on total area 2011 (P/Ha)	Type of Land Use from GIS map				Eq. Area (Ha)	Present Population Density on Eq. area	Density		Iteration=1		Iteration=2		Iteration=3	
				Residential (Ha)	Public* Area (Ha)	Industrial** (Ha)	No Man's Area *** (Ha)			Main Category	Sub-category	Projected density for design year 2046 (p/Ha)	Projected population for design year 2046	Projected density for design year 2046 (p/Ha)	Projected population for design year 2046	Projected density for design year 2046 (p/Ha)	Projected population for design year 2046
3	14.4	974	68	13.1	1.2	0	0	13.44	72.46	High	H2	120	1613	110	1479	110	1479
7	12.8	1169	91	12.8	0	0	0	12.79	91.39	High	H2	120	1535	110	1407	110	1407
6	18	1379	76	12.3	5.5	0.3	0	13.66	100.97	Saturated	S	146	1994	146	1994	147	2008
4	11.4	1616	142	11	0.2	0.2	0	11.04	146.34	Saturated	S	146	1612	146	1612	147	1623
		30450		793	129	3	9	825					46020		41946		40608

* Public : This includes Commercial, Park Playground, Public & Semi-public, and Public Utility
** Industrial : This includes Agriculture, industrial, playground
*** No Men's Area : This includes Roads and Water Body
# The main category is further divided into a subcategory. For example, if the main category is 0-40 (P/Ha), then subcategory L1 is 0-25, and L2 is 25-40 (P/Ha)

Why is Equivalent Area Using General Factors?

It is necessary to reach a methodology that can be easily adopted, realistic, and plans for future demand. It should ultimately result in the node-wise allocation of demands for making the design of the water network for future realistic demand. The methodology presented here is for an urbanised town, which essentially comprises of mainly residential areas and other land uses such as commercial, public, agricultural, etc. These land uses are categorised into four categories.

Factor for residential land use: Consumption of drinking water is for all the people living in a residential area. This category also includes an area having shops on the ground floor and residential flats on the above floors (which may also include minor commercial activity). In short, this category is having a maximum area of the town. For this category land use factor is 1 (i.e., 100%).

Factor for No Men's Area: This category includes water bodies, the area covered under roads, etc. Since human consumption is nil, the land use factor for this category is zero.

Factor for Public Area: This category of land use includes public utilities like schools, colleges, offices, and single-story market yards but excludes hospitals and lodges. The human consumption of such areas on a hectare basis is low compared to residential areas as the human population in such areas is not expected to take a bath, cook food, etc., as compared to the residential area. Hence, 25% of the total such area would be comparable with residential area for computation of water demand; thus, the land use factor shall be considered as 0.25 for such areas.

Factor for Agriculture Area: This includes areas with agricultural land, playgrounds, and industrial areas with fewer workers but big areas. Such areas will need very low water demand per hectare of land as human water consumption is very low. However, it should not be forgotten that the demand so calculated is for human consumption only and does not include the actual requirement of water for agriculture/horticulture or industrial. Hence, 10% of the total such area would be comparable with residential area for computation of water demand; hence, the land use factor shall be considered as 0.1 for such areas. Thus,

$$\begin{aligned}\text{Equivalent Area} = {} & (\text{Residential Area x 1}) + (\text{Public Area x 0.25}) \\ & + (\text{Agricultural Area x 0.1}) + (\text{No Men's Area x 0}) \end{aligned} \tag{1}$$

The demand for hospitals and lodges should be worked out through a consumer survey and should be extrapolated for the future, and should be shown as point loads, in addition to demand determined on equivalent area basis.

Present Population Density: Present Population density is a ratio of the present population of each ward to its equivalent area. These are computed in Column 10 of Table 1. The next step is to formulate a method to project future ward-wise populations on a per hectare basis. This method ought to be realistic and easy to compute.

Methodology for Projecting Future Population

Future Population Density: It is described as below:

Use of CDP:

Many progressive Municipal Corporations/Councils have already prepared their own CDP showing ward-wise area, present population, and stage-wise projected future population for such wards. But most of the CDP maps of cities do not have ward-wise land use areas in them, and they only provide a map of the total city showing different types of land use areas in different colour coding. Now it is necessary to compute land use areas for each ward as discussed above. And then, using the equivalent area method, the future population for each ward should be computed.

The areas which are already fully developed, such as the cores of the city, the density of these areas has already practically reached to the level of saturated density unless Floor Space Index (FSI) is increased. It is recommended not to increase FSI indiscriminately and as far as possible, keep the present FSI unchanged. For accommodating a small increase in the future, the saturation density should be decided very close to the present density. Eventually, this density, called as saturation density, will not be surpassed in any other area of the city. In other words, the saturation density thus determined is an upper limit for the future density of other areas. For other areas, assign future densities based on the expected new layouts, vertical growth, urban poor, slums, land use pattern, residential and commercial properties, industries, etc. The areas/wards closer to the present cores of the city, i.e., areas having saturation population grow faster compared to the areas which are away. The areas which are beyond a barrier, such as water bodies, railway tracks, etc., grow slower. Where the price of the land is expected to increase faster, the population in that area/ward is also expected to grow faster.

The rational method presented here is an iterative (Figure 5) and is as follows:

Sort the ward-wise present population densities based on the equivalent area from the minimum density at the top of the table to the maximum density at the bottom of the table, as shown in Column 10 of Table 1.

It is necessary to divide the sorted present densities into four categories: low, medium, high, and saturated. In the case of the town under consideration, the ranges chosen are 0-31 people per hectare for low; 31-45 people per hectare for medium; 45-95 people per hectare for high, and more than 95 people per hectare for saturated densities. Values mentioned in these ranges depend upon the size of the town, and the more the size, the greater is the range.

For understanding the growth pattern in terms of future population density, work out the ratio of the future population to the present population. In the case of the town considered, the present population for the year 2011 is 30,450, and the future projected population for the year 2046 is 40,631; hence, the ratio works out to 1.334. The total equivalent area for the town is 825 hectares. Present population density (on equivalent area) for the total town works out to 36.9 (=30,450/825) persons per hectare, and the future population density (on equivalent area) for the total town is 49.2 (=40,631 /825) persons per hectare. Eventually, the increase in density is also 1.334 (=49.2/36.9) times.

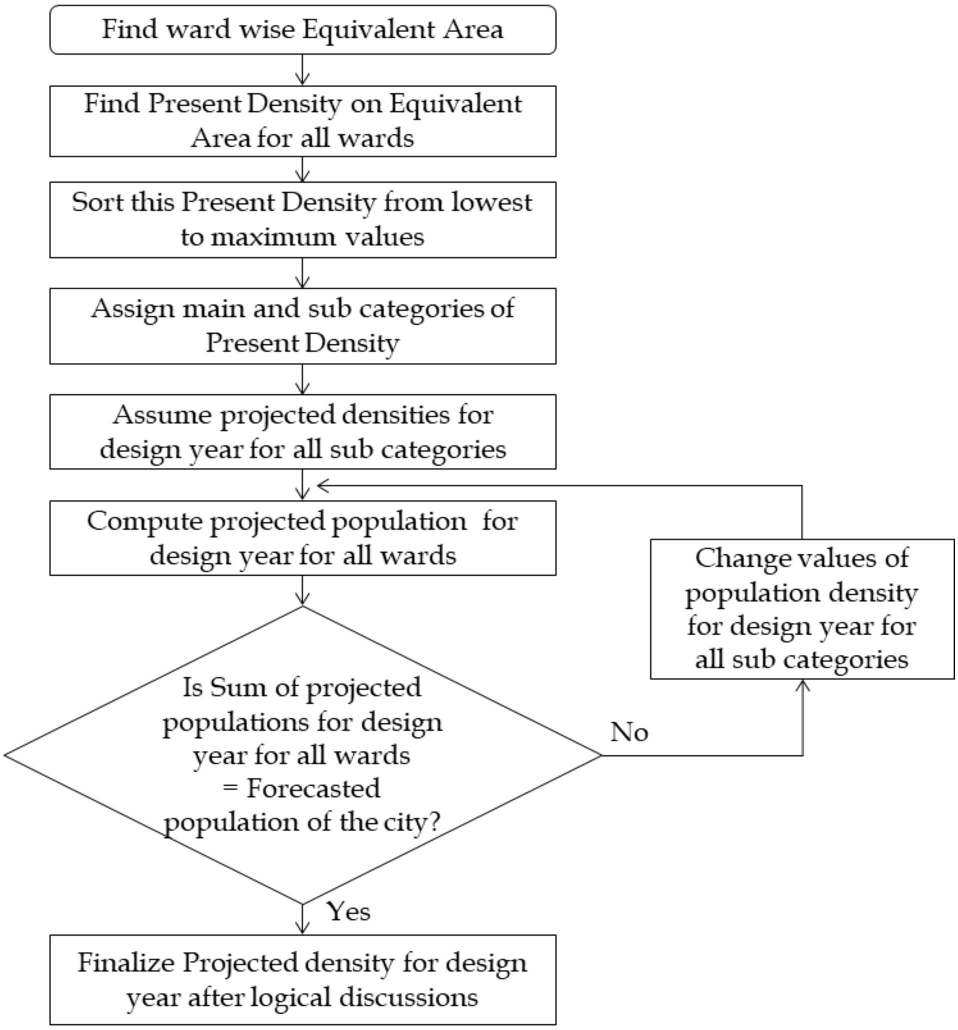


Figure 5: Process of computation of future population densities for each ward

Based upon the saturation density and present low density, decide the slabs of future densities. Choose one of the slabs for each ward. For this town, the present population density for the total town on a total equivalent area is 36.9 persons per hectare, and the future density is 49.2 persons per hectare. The saturation density for this town is 146 persons per hectare, as shown in the bottom row of Table 1. Hence, the slabs to be chosen are from (13.57×1.334 = 18.1) 25 to 146 persons per hectare. Appropriate population density needs to be now assumed in Column 13 for each of the subcategories which are shown in Column 12 of Table 1. In the first iteration (as shown in Columns 13 and 14 of Table 1), the sum of the future population for all 23 wards is computed as 46,020, which is found to be more than the future projected population of 40,631. Thus, the slab-wise population densities, assumed in the first iteration, need now to be reduced. The exercise has been carried out in the second iteration, in Columns 15 and 16 of Table 1, and the total population works out to 41,946, which is still slightly more than the future projected population of 40,631. Hence, the third iteration is carried out in Columns 17 and 18 of Table 1, and the total population now works out to 40,608, which fairly tallies with the total estimated population of 40,631. In this way, the future population density and future population in various stages are finalised for every ward.

The future ward-wise population density thus determined in Table 1 should be discussed with logical discussions with technocrats, bureaucrats, and a few knowledgeable people representatives. Based on the outcome of the discussions with them, the future population and population densities of each ward should be finalised. While finalising, it should not be forgotten that the total population thus finalised must tally with the projected population of the whole town. Ward-wise future population and population density thus determined should be approved by the local body, and then only the engineer/consultant should be allowed to proceed with detailed engineering designs and estimation.

Mapping Future Population Density:

Resulting Table 1 shows ward numbers and ward-wise future population densities in the non-spatial Excel format. Table 1 should be added to the GIS software and joined with the GIS map. Thus, a shapefile of the final population density map is prepared (Figure 4). This shapefile is important because it is used to allocate the demands to each node of the network of the distribution system.

Annexure 2.8: Population Forecast Methods

Methods of Population Projection

Demographic Method

Population change can occur only in three ways (i) by births (population gain), (ii) by deaths (population loss), or (iii) by migration (population loss or gain depending on whether movement out or movement in). Annexation of an area may be considered as a special form of migration. Population forecasts are frequently obtained by preparing and summing up of separate but related projections of natural changes and net migration and are expressed as below:

The net effect of births and deaths on population is termed natural increase (natural decrease if deaths exceed births).

Migration also affects the number of births and deaths in an area, so projections of net migration are prepared before projections for natural increase.

This method thus considers the prevailing and anticipated birth rates and death rates of the region or city for the period under consideration. An estimate is also made of the emigration from and immigration to the city, growth of city area wise, and the net increase of population is calculated accordingly considering all these factors, by arithmetical balancing.

Arithmetical Increase Method

This method is suitable for a large, well-settled city with considerable development. If it is used for small, average, or comparatively new cities, it may give comparatively lower predictions. In this method, the average increase in population per decade is calculated from the past census reports. This increase is added to the present population to find out the population for the next decade. Thus, it is assumed that the population is increasing at a constant rate. Thus,

Population after nth decade will be $Pn = P + n*Xav$ Eq. (2.1)

Where Pn is the population after n decade, P is the present population, and Xav is the average increase in population per decade.

Incremental Increase Method

This method is the modification of the arithmetical increase method. It is suitable for an average size town under the normal condition where the growth rate is found to be in increasing order. This method does not take into account local or government initiatives that may drive the growth in population. Hence, due care must be taken while considering population projection based on this method.

While adopting this method, the increase in arithmetic increment is considered for calculating the future population. The incremental increase is determined for each decade from the past population, and the average value is added to the present population along with the average rate of increase. In some cases, population forecast by this method gives forecasted population less than present population due to negative value of incremental increase (Yav). In such cases, this method may be discarded.

Population after n^{th} decade will be

$$P_n = P + nX_{av} + \left\{\frac{n(n+1)}{2}\right\} \times Y_{av} \qquad \text{Eq. (2.2)}$$

Where,

P = present population

Pn = Population after nth decade

Xav = Average increase

Yav = Incremental increase

Geometrical Increase Method

In this method, the percentage increase in population from decade to decade is assumed to be the rate of growth, and the geometric mean of percentage increases is used to find out the future increment in population. This method can be applied to growing towns and cities having lots of scope for expansion, for example, a new industrial town at the beginning of development for only a few decades. One of the major disadvantages of this method is that if one of the observations is negative, the geometric mean will be imaginary, despite the other set of observations. Another disadvantage is that this method gives excessive population projection which may only be acceptable for the areas which see population explosion.

The population at the end of nth decade 'P_n' can be estimated as

$$P_n = P\left\{1 + \frac{r_g}{100}\right\}^n \qquad \text{Eq. (2.3)}$$

Where

P = Present population

n = number of decades

r_g = geometric mean (%), i.e., n^{th} root of ($r1*r2*r3...r^n$)

Or,

$$r_g\ (\%) = (r_1 * r_2 * r_3 * r_4 \dots . r_n)^{1/n} \qquad \text{Eq. (2.4)}$$

r = percentage growth rate = (increase in population/initial population) * 100

Step 1: Find an increase in population for each decade

Step 2: Find the growth rate for each decade

Step 3: Find the number of decades (n) between last known year and the required year

Step 4: Find average growth rate (r_g) using geometric mean

Step 5: Apply the formula (eq. 2.3)

Decreasing Rate of Growth Method

In this method, it is assumed that the rate of percentage increase decreases over time, and the average decrease in the rate of growth is calculated. Then the percentage increase is modified by deducting the decrease in the growth rate. This method is applicable in the cases where the rate of growth of the population shows a downward trend.

Step 1: Find the increase in population.

Step 2: Find the growth rate (r) as in the geometrical increase method.

Step 3: Find the decrease in the growth rate.

Step 4: Find the average of decrease in growth rate(s).

Step 5: Apply the formula.

$$P_n = P_{(n-1)} + \left\{\frac{r_{(n-1)} - S}{100}\right\} P_{(n-1)} \qquad \text{Eq. (2.5)}$$

Where,

P_n = population at required decade,

$P_{(n-1)}$ = population at previous decade (predicted or available),

$r_{(n-1)}$ = growth rate at previous decade and,

S = average decrease in growth rate

Note: The formula requires population data from the previous decade, i.e., P(n-1). Thus, this method requires the calculation of population at each successive decade (from the last known decade) instead of directly calculating the population at the required decade.

Graphical Method

In this approach, there are two methods. In one, only the city in question is considered, and in the second, other similar cities are also taken into account.

Graphical Method Based on Single City

In this method, the population curve of the city (i.e., the Population vs. Past Decades) is smoothly extended to get future value. This extension has to be done carefully, and it requires vast experience and good judgment. The line of best fit may be obtained by the method of least squares.

Graphical Method Based on Cities with Similar Growth Pattern

In this method, the city in question is compared with other cities which have already undergone the same phases of development that the city in question is likely to undergo, and based on this comparison, a graph between population and decades is plotted.

Logistic Method

This method is used when the growth rate of population due to births, deaths, and migrations takes place under normal situation and it is not subjected to any extraordinary changes like an epidemic, war, earthquake or any natural disaster, etc. the population follow the growth curve characteristics of living things within limited space and economic opportunity. If the population of a city is plotted with respect to time, the curve so obtained under normal conditions looks like an S-shaped curve and is known as a logistic curve.

Method of Density

In this approach, a trend in the rate of density increase of population for each sector of a city is found out, and a population forecast is done for each sector based on the above approach. The addition of sector-wise population gives the population of the city.

Curvilinear Method

In this method, it is assumed that the population of a city will grow in the same manner as in other cities in the past. This similarity between the cities includes geographical proximity, the similarity of economic base, access to similar transportation systems, etc.

Floating Population

The floating population is to be considered in the areas with daily/seasonal employment and tourist influx. It should be calculated by estimating the market potential of the city/town/village. Population indicators like utility consumption can be used for the transient census. The floating population should be got certified by the Chief Officer/Deputy Commissioner of ULB. Data from Tourist Bureau, Check/ Entry Tax points, Mandi associates, etc., should be obtained and extrapolated/projected. In absence of floating population data, it should be taken as 1-3% of base population judiciously considering the tourist inflow.

Final Forecast

While the forecast of the prospective population of a projected area at any given time during the period of design can be derived by any one of the foregoing methods appropriate to each case, the density and distribution of such population within the several areas, zones, or districts will again have to be evaluated with a discerning judgment on the relative probabilities of expansion within each zone or district, according to its nature of development and based on existing and contemplated town planning regulations.

Wherever population growth forecast, or master plans prepared by town planning or other appropriate authorities are available, the decision regarding the design population may consider their figures. Using the methods discussed above, we make a population forecast of a city.

Distribution of Forecasted Population: After deciding the total population of the city by using the above methods, it is important to distribute it ward-wise so that the total demand is realistically determined. As pipelines are to be designed for ultimate stage population, distribution of ultimate stage population is required for getting realistic diameters of the proposed distribution system. Distribution of immediate stage population is required for getting realistic capacities of service tanks that are constructed for the immediate stage.

An illustrative example of the population forecast of a city is resented below:

Illustrative Example: Consider a town whose population figures as per census records are given in Table 1 for the decades 1971 to 2011. Assuming that the water supply scheme will commence to function from 2025, it is required to estimate the population of 30 years, i.e., the present stage in the year 2025, an intermediate stage in the year 2040, and the ultimate stage in the year 2055.

Solution: Increase in population per decade, the total increase in population from the year 1971 to the year 2011, and an average increase in population (X) per decade are shown in column 3 of Table 1. Incremental increase per decade and average incremental increase (Y) are calculated and shown in Column 4 of Table 1. The rate of growth (r) per decade is calculated and given in Column 5. The geometric mean is calculated from the equation (2.4). The number of decades for =different stages is shown in Table (2).

Table 1: Census data

Year	**Population**	**Increment X**	**Incremental Increase-Y**	**Rate of growth per decade**
1	2	3	4	5
1971	27,279			
1981	39,068	11,789		(11,789/27,279)*100 = 43.2%
1991	68,019	28,951	17,162	(28,951/39,068)*100 = 74.1%
2001	80,625	12,606	-16,345	(12,606/68,019)*100 = 18.5%
2011	112,085	31,460	18,854	(31,460/80,625)*100 = 39.02%
Average	65,415	21,202	6,557	
				rg= 39.01%

Arithmetic Increase method

Step 1: Find the increase (X) in population each decade.

It is computed in Col 3 of Table 1.

Step 2: Find the average rate of increase of population (Xav)

Xav = (11,789 + 28,951 + 12,606 + 31,460)/4 = 21,202

Step 3: Find the number of decades (n) between the last known year and the required year.

Table 2: Number of decades for different stages

Sr. No.	Year	Number of years	Number of Decade
1	2025	14 (=2025-2011)	1.40
2	2040	29 (=2040-2011)	2.90

3	2055	44 (=2055-2011)	4.40

Step 4: Apply the formula (Eq. 2.1) Pn= P + n*Xav

Therefore, population in the year 2025, 2040 and 2055 are as below:

P(2025) = P(2011) + 1.4 (21,202) = 1,12,085+1.4(21,202)= 141,768

P(2040) = P(2011) + 2.9 (21,202) = 1,12,085+2.9(21,202)= 173,571

P(2055) = P(2011) + 4.4 (21,202) = 1,12,085+4.4(21,202)= 205,374

Incremental Increase Method

Step 1: Find the increase in population in each decade.

It is computed in Col 3 of Table 1.

Step 2: Find the incremental increase, i.e., increase of increase.

It is computed in Col 4 of Table 1.

Step 3: Find Xav and Yav as average of Increase in population and Incremental increase values, respectively.

Xav = 21,202

Yav= {17,162 + (-16,345) +18,854}/3 = 6,557

Step 4: Find the number of decades (n) between the last known year and the required year.

It is computed in Table 2.

Step 5: Apply the formula (Eq. 2.2) $P_n = P + nX_{av} + \left\{\frac{n(n+1)}{2}\right\}\ x\ Y_{av}$.

P(2025) = 112,085 + 1.4(21,202) + {1.4(1.4+1)/2} (6557) = 152,783

P(2040) = 112,085 + 2.94(21,202) + {2.9(2.9+1)/2} (6557) = 210,650

P(2055) = 112,085 + 4.4(21,202) + {4.4(4.4+1)/2} (6557) = 283,271

Geometric Progression method

Step 1: Find the increase in population each decade.

It is computed in Col 3 of Table 1.

Step 2: Find the growth rate.

It is computed in Col 5 of Table 1.

Step 3: Find the geometric mean (r) using the equation,

rg (%) = $(r_1 * r_2 * r_3 * r_4 \ldots . r_n)^{1/n}$

$Rg=(43.216*74.104*18.533*39.020)^{(1/4)} = 39.01\%$

Step 4: Find the number of decades (n) between the last known year and the required year.

It is computed in Table 2.

Step 5: Apply the formula $P_n = P\left\{1 + \frac{r_g}{100}\right\}^n$

$$P(2025) = 112085 \left\{1+\frac{0.39}{100}\right\}^{1.4} = 1,77,752$$

$$P(2040) = 112085 \left\{1+\frac{0.39}{100}\right\}^{2.9} = 2,91,330$$

$$P(2055) = 1,12,085 \left\{1+\frac{0.39}{100}\right\}^{4.4} = 4,77,482$$

Note: If, in a given year, the value of the rate of growth per decade (r) is observed to be zero or negative, then that value of population and rate of growth per decade (r) may be neglected.

Decreasing Rate of Growth Method

Find the population for the years 2025, 2040, and 2055.

Step 1: Find the increase in population.

It is computed in Col 3 of Table 3.

Table 3: Decrease in growth rate

Year	Population	Increase in population	Rate of growth per decade	Decrease in growth rate
1	2	3	4	5
1971	27,279			
1981	39,068	11,789	43.216	
1991	68,019	28,951	74.104	74.104-43.216 = 30.89%
2001	80,625	12,606	18.533	18.533-74.104 = (-)55.57%
2011	112,085	31,460	39.02	39.02-18.533 = 20.49%

Step 2: Find the growth rate (r) as in the geometrical increase method.

It is computed in Col 4 of Table 3.

Step 3: Find the decrease in the growth rate.

It is computed in Col 5 of Table 3.

Step 4: Find the average of decrease in growth rate(s).

S = (30.89-55.57+20.49)/3= -1.40%

Step 5: Apply the formula $P_n = P_{(n-1)} + \left\{\frac{r_{(n-1)} - S}{100}\right\} P_{(n-1)}$

Where

Pn = population at required decade

P(n-1) = population at previous decade (predicted or available)

r(n-1) = growth rate at previous decade

S = average decrease in growth rate

Note: The formula has a typical nature. Hence, it requires population data from the previous decade, i.e., P(n-1). Thus, this method requires the calculation of population at each successive decade (from the last known decade) instead of directly calculating the population at the required decade.

Find the population at successive decade till the population at required data is arrived.

Table 4: Population at Successive Decades

Year	Net percentage increase in population	Population
2021	39.02 - 1.40 = 37.620	112,085 + (37.620/100)*112,085 = 154,251
2031	37.62 - 1.40 = 36.22	154,251+ (36.22/100)*154,251 = 210,121
2041	36.22 - 1.40 = 34.82	210,121+ (34.82/100)*210,121 = 283,285
2051	34.82 - 1.40 = 33.42	283,285 + (33.42/100)*283,285 = 377959
2061	33.42 - 1.40 = 32.02	377,959 + (32.02/100)*377,959 = 498,981

The population of 2025, 2040, and 2055 can be computed by interpolation, which is shown in Table 5.

Table 5: Interpolated population

Year	Population
2025	175,000
2040	275,000
2055	423,000

Annexure 2.9: Disaster Management when Source Fails

Mumbai Metropolitan Area, including Mumbai, India's financial capital, received a record-breaking 942 millimetres of rain in a 24-hour period on 26 July 2005. The heavy monsoon rain triggered off deadly floods, which have claimed as many as 418 lives in Mumbai alone. The metropolis was practically cut-off from other parts of India. Railway tracks, which are the city's lifelines, were washed away. The plight of Mumbai's citizens worsened as telephone services, including mobile phone networks, crashed. The rains were so devastating that the water supply schemes were disrupted, and the water supply of suburban towns was totally affected. Some of the WTPs were inundated, and the pumps and transformers were submerged. The electric power system was severely affected. Events were so sudden that they were highly disruptive and had created an inability on an organisation's part to provide critical water supply in the initial few hours.

It was a major challenge to provide a critical water supply immediately. All the water supply departments swung into action and were able to restore most of the water supply of the cities except for Ambernath and Badlapur cities, whose common source was ruined due to the washing of the gates over the barrage. The water supply of these cities and that of Panel was restored by switching it to other sources. Considering the success of this type of arrangement, an idea of interlinking of all water supply systems in the Mumbai metropolitan area has come forward and was implemented.

Interlinking of all the water supply systems of 12 cities in the Mumbai metropolitan area ensured that if a source of the water supply of one of the cities gets affected, then water can be supplied to the affected city by an alternate source.

The objective, therefore, is to present methodology and a model that describes a simulation of behaviour, i.e., a hydraulic model of the systems, and how it would help in restoring water supply whenever disaster strikes.

1. Collection of Data and Simulation

A vast data describing a real-world network system was collected from all waterworks before building a model. One foremost difficulty faced is that there was no single comprehensive drawing showing all the water supply systems. The preparation of such an all-inclusive map (integration of water supply schemes in the region) was a daunting task. Fortunately, a drawing showing all the metropolitan boundaries of the Mumbai area was available with Maharashtra Jeevan Pradhikaran (MJP), which was tested for accuracy and was corrected and synchronised with the GPS system. All the sources, such as impounding dams and rivers, were shown in this drawing. Water transmission pipelines were then drawn, and the positions of WTPs, master balancing tanks, and the ESRs in the city were digitised on the GIS drawing. This drawing was then used as a backdrop for the network software. While working on the model, this backdrop was extremely useful as an interface for referring to the real system. The various tasks that were performed to assemble the model are shown in Figure 1.

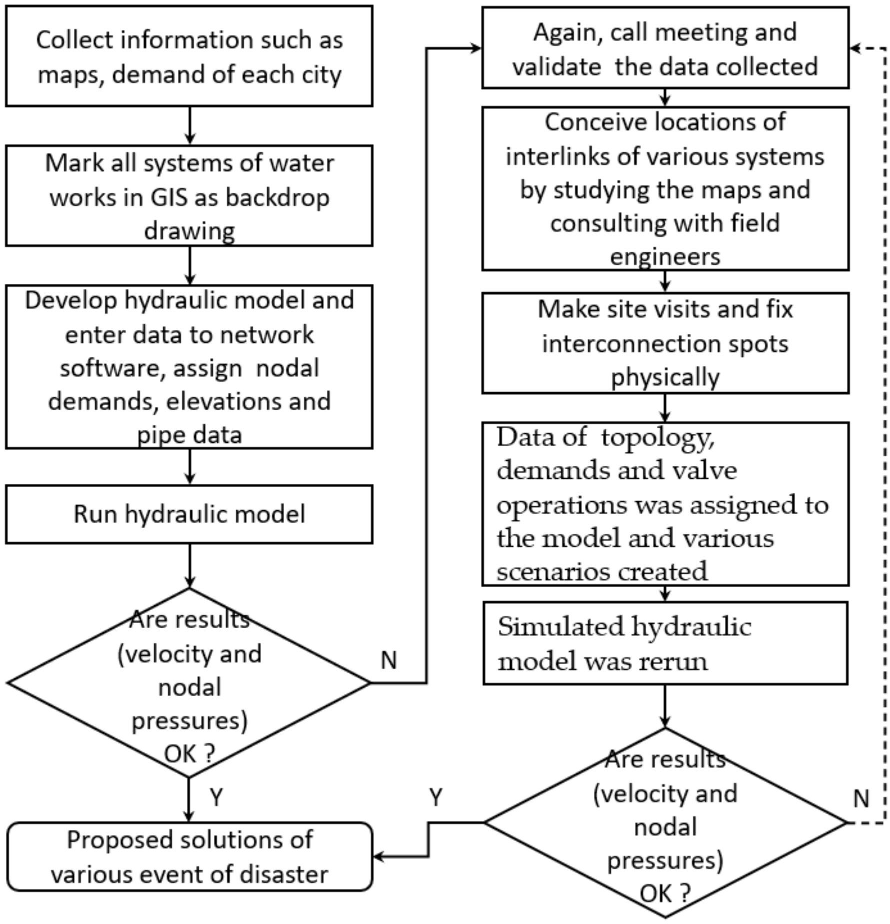


Figure 1: Modelling process.

Here, a simulation of the network was used to imitate the dynamics of existing and proposed networks in the metropolitan area of Mumbai, which helped to predict the system response in the event of a disaster. Using the concept of simulation, the problems of distributing water in crisis were anticipated well in advance that might arise in the existing and proposed networks in the area. It helped to find solutions to the problems in the disaster conditions and saved time, money, and other resources.

2. Simulation of Failure

Principle of Simulation: The principle of simulation of interlinking of waterworks is depicted in Figure 2 (a), (b), and (c). In Figure 2 (a), under normal water supply, sources 1 and 2 are shown supplying water to the city through transmission pipelines of T1 and T2. Since the sources are intact (unaffected), all parts of the city get a normal water supply.

Out of these two sources supplying water to the city, if source 1 fails, as shown in Figure 2 (b), then some parts of the city get no water. In Figure 2 (c), the interlinking of the two sources 1 and 2 is shown. Here, source 2 is interlinked with the disrupted source 1. Therefore, customers of source 1 get at least some quantity of water. However, the water supply of customers of source 2 is slightly depleted.

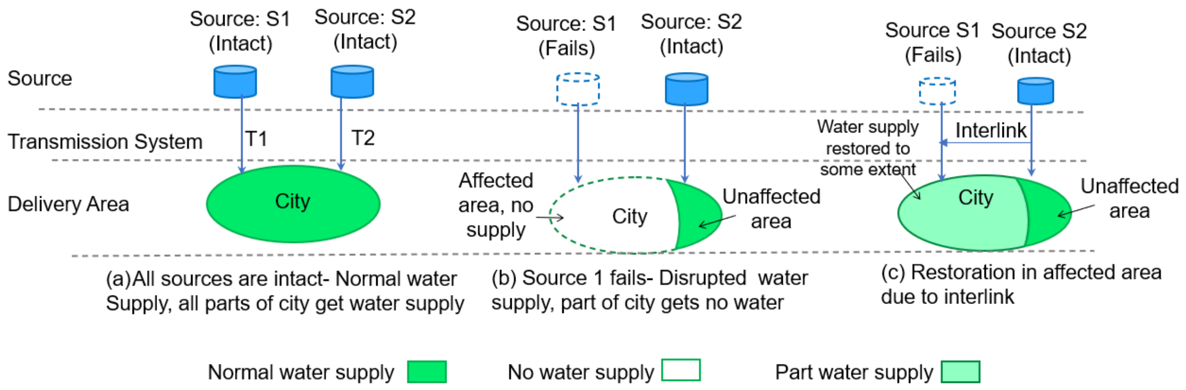


Figure 2: Principle of simulation

It is expected that the customers of the affected source must get at least 30% of their normal water supply in the period of crisis. To simulate this condition, the transmission network of sources 1 and 2 are joined together by the interlinked pipeline. By doing this here, a relationship has been established between transmission system T1 of source S1 and transmission system T2 of the source S2 of the hydraulic model. Now the nodal demands of the transmission network of the source S1 are modified, their values are corrected to 30% of their normal demands, and the combined network is hydraulically solved so that the customers of this source now get at least 30% water supply.

This principle has been used in the study to simulate all the transmission networks of the water works in the Mumbai metropolitan area.

3. Application of Model for Interlinking of Water Supply Systems

During the disaster of 26July 2005, interlinking of water supply systems of Ambernath and Badlapur was carried out.

Ambernath and Badlapur Water Works: Badlapur barrage (Figure 3) supplies water to both the cities of Badlapur and Ambernath. Badlapur city gets 23 MLD, Ambernath 36 MLD, and the Ordinance factory gets 13 MLD water from this barrage. The barrage had old gates which were erected by the Britishers before independence. These gates impounded water before the disaster. During the floods of 26July 2005, these gates were washed away, and there was no storage of water.

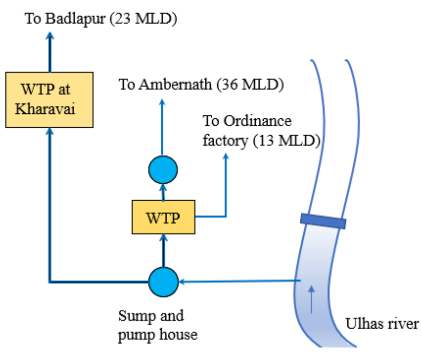


Figure 3: Waterworks at Barrage

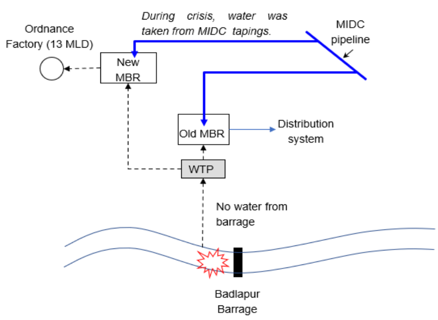


Figure 4: Transmission system of Ambernath city during crisis.

There was an urgent need to supply more water. Immediately. The collector summoned 50 tankers from Vasai-Virar to cater to the needs of water supply. The tension was quite high as within a couple of days; there would have been a law-and-order situation in both the cities.

The only solution that was possible was to take water from adjoining MIDC pipelines which were fortunately laid along the periphery of both the cities.

Interlinks: Two tapings were taken from MIDC pipelines. 45 MLD water from MIDC pipelines was transmitted to the new MBR and old MBR, and then it was distributed to Ambernath city, the ordnance factory, and the west part of the Badlapur city. Cross-connections taken from the MIDC pipeline are shown in Figures 4 and 5.

(a) 300 mm connection taken from MIDC pipeline

(b) MIDC pipeline joined to MJP pipeline

Figure 5: Cross-connections taken from MIDC pipeline

4. Simulation of Network in Crisis

Minimum crisis demands of 30% and the scenario of this crisis condition were created using the hydraulic model to which the datasets of topology, demand, and operational were attached.

In normal conditions, the MJP system provides a total of 72 MLD water to the Balapur, Ambernath cities, and Ordinance factory. During the time of crisis, it is expected that both the cities will get at least 30% of its normal water supply, and the ordinance factory will get 100% supply. Accordingly, the demand for various nodes is assigned in the hydraulic model. Various demands are shown in Table 1.

Table 1: Various demands during normal and disaster conditions

Requirement	Badlapur	Ambernath	Ordnance factory	Total
Daily Requirement (MLD)	23	36	13	72
30% Requirement in crisis period (MLD)	6.9	10.8	3.9	21.6
MIDC actually supplied (MLD)	10	22	13	45
Percentage (%)	43%	61%	100%	63%

FCVs are used to regulate flow in the model of the pipe network. When interconnections of pipes are made to divert the flow of 45 MLD from the MIDC system to the MJP system, FCVs are installed to regulate the flow through the interconnection pipe. In such a situation, the dataset of operational is very important which is used.

After assigning three datasets to the scenario of minimum crisis demand of 30%, the model was run and results were compared for normal water supply, minimum 30% demand, and actual demand in the disaster period. It is observed that even after interlinking water works, affected areas get water with adequate pressures. Though a minimum of 21.6 MLD was required, MIDC provided 45 MLD water which had given greater relief.

On experiencing the success of this work in a disaster, Govt. of Maharashtra then constituted one committee that has created interlinks for all the cities in Mumbai Metropolitan Area. This comprehensive action plan is then prepared for tackling any failure of sources in the future.

5. Conclusions

Using present model, all 14 water supply systems of the Mumbai metropolitan area are studied for preparing a disaster mitigation plan. This plan was rehearsed periodically in all water supply systems. Proper training should be imparted to the operators regarding the valve operations during a disaster. After interlinking waterworks, it must be ensured that the affected areas get water with adequate pressure. Water grid which was implemented in Mumbai metropolitan area linking all the water utilities, various departments responsible for water supply could respond to emergencies as a group and save citizens from needless anxiety and suffering.

Annexure 4.1: State-Wise Depth to Water Level

Number and Distribution of Percentage of Wells for the Period of November-2021 in Unconfined Aquifer

S. No.	Name of State/UT	No. of wells Analysed	Depth to Water Level (mbgl)		Number & Percentage of Wells Showing Depth to Water Level in metres below ground level (mbgl) in the Range of											
					0–2		2–5		5–10		10–20		20–40		>40	
			Min	Max	No	%	No	%	No	%	No	%	No	%	No	%
1	Andaman and Nicobar	94	0.07	7.04	82	87.2	10	10.6	2	2.1	0	0.0	0	0.0	0	0.0
2	Andhra Pradesh	724	0.00	49.99	367	50.7	225	31.1	98	13.5	27	3.7	6	0.8	1	0.1
3	Arunachal Pradesh	10	1.25	6.80	1	10.0	8	80.0	1	10.0	0	0.0	0	0.0	0	0.0
4	Assam	169	0.02	16.67	65	38.5	84	49.7	17	10.1	3	1.8	0	0.0	0	0.0
5	Bihar	593	0.00	10.69	232	39.1	326	55.0	33	5.6	2	0.3	0	0.0	0	0.0
6	Chandigarh	12	2.80	51.31	0	0.0	3	25.0	1	8.3	4	33.3	3	25.0	1	8.3
7	Chhattisgarh	694	0.31	22.00	110	15.9	390	56.2	162	23.3	29	4.2	3	0.4	0	0.0
8	Dadra and Nagar Haveli	17	1.17	7.05	4	23.5	11	64.7	2	11.8	0	0.0	0	0.0	0	0.0
9	Daman and Diu	5	0.14	6.35	1	20.0	2	40.0	2	40.0	0	0.0	0	0.0	0	0.0
10	Delhi	87	0.14	65.67	15	17.2	18	20.7	23	26.4	15	17.2	8	9.2	8	9.2
11	Goa	68	1.10	15.70	6	8.8	32	47.1	24	35.3	6	8.8	0	0.0	0	0.0
12	Gujarat	747	0.00	52.33	170	22.8	276	36.9	177	23.7	88	11.8	33	4.4	3	0.4
13	Haryana	183	0.63	76.50	14	7.7	40	21.9	34	18.6	50	27.3	35	19.1	10	5.5
14	Himachal Pradesh	86	0.24	32.85	23	26.7	29	33.7	18	20.9	14	16.3	2	2.3	0	0.0

S. No.	Name of State/UT	No. of wells Analysed	Depth to Water Level (mbgl)		Number & Percentage of Wells Showing Depth to Water Level in metres below ground level (mbgl) in the Range of											
					0–2		2–5		5–10		10–20		20–40		>40	
			Min	Max	No	%	No	%	No	%	No	%	No	%	No	%
15	Jammu and Kashmir	213	0.32	31.80	79	37.1	80	37.6	39	18.3	9	4.2	6	2.8	0	0.0
16	Jharkhand	205	0.35	10.09	26	12.7	145	70.7	33	16.1	1	0.5	0	0.0	0	0.0
17	Karnataka	1,290	0.01	26.58	502	38.9	454	35.2	287	22.2	46	3.6	1	0.1	0	0.0
18	Kerala	1,304	0.01	53.60	330	25.3	459	35.2	425	32.6	83	6.4	6	0.5	1	0.1
19	Madhya Pradesh	1,296	0.08	37.76	168	13.0	588	45.4	422	32.6	105	8.1	13	1.0	0	0.0
20	Maharashtra	1,757	0.01	54.70	512	29.1	839	47.8	315	17.9	71	4.0	18	1.0	2	0.1
21	Meghalaya	24	0.20	4.80	10	41.7	14	58.3	0	0.0	0	0.0	0	0.0	0	0.0
22	Nagaland	2	3.58	6.05	0	0.0	1	50.0	1	50.0	0	0.0	0	0.0	0	0.0
23	Odisha	1,253	0.06	13.60	503	40.1	602	48.0	139	11.1	9	0.7	0	0.0	0	0.0
24	Puducherry	6	0.83	2.50	4	66.7	2	33.3	0	0.0	0	0.0	0	0.0	0	0.0
25	Punjab	176	0.02	42.00	13	7.4	43	24.4	34	19.3	48	27.3	36	20.5	2	1.1
26	Rajasthan	919	0.25	116.98	59	6.4	171	18.6	185	20.1	177	19.3	162	17.6	165	18.0
27	Tamil Nadu	541	0.05	62.50	230	42.5	142	26.2	113	20.9	42	7.8	7	1.3	7	1.3
28	Telangana	545	0.00	50.14	154	28.3	252	46.2	100	18.3	29	5.3	9	1.7	1	0.2
29	Tripura	22	0.31	6.58	6	27.3	13	59.1	3	13.6	0	0.0	0	0.0	0	0.0
30	Uttar Pradesh	650	0.00	43.95	233	35.8	202	31.1	115	17.7	81	12.5	16	2.5	3	0.5
31	Uttarakhand	45	0.04	55.16	7	15.6	11	24.4	15	33.3	6	13.3	4	8.9	2	4.4
	Total	**14,470**	**0.00**	**116.98**	**4191**	**29.0**	**5755**	**39.8**	**2923**	**20.2**	**1011**	**7.0**	**384**	**2.7**	**206**	**1.4**

Source: Groundwater Yearbook (CGWB), India 2021-22

Annexure 4.2: Application for Issue of NOC to Abstract Ground Water

Annexure 4.3: Yield Test for Wells

General

Pumping tests are made on wells to determine their capacity and other hydraulic characteristics and to obtain information so that permanent pumping equipment can be intelligently selected. Preliminary tests of well drilled as test holes are sometimes made to compare yielding ability of different water bearing formation or different locations in same formation. This information is then used as a basis for selecting the best site for a supply well and the aquifer in which it should be completed.

Measurements

The measurement that should be made in testing wells include the volume of water pumped per minute or per hour, the depth to the static water level before pumping is started, the depth to the pumping level at one or more constant rates of pumpage, the recovery of water level after pumping is stopped and the length of time the well is pumped at each rate during test procedure.

Pumping Procedure

The pump and power unit used for testing a well should be capable of continuous operation at a constant rate of pumpage for several hours. It is important that the equipment be in good condition for an accurate test, since it is not desirable to have a shut down during the test. If possible, the test pump should be large enough to test the well beyond the capacity at which it will eventually be pumped, but this may not be always practicable under field operations.

In the pumping test, the pump is fixed close to the well and water is pumped out. The quantity of water pumped is measured using a circular orifice metre or a V notch. The water discharging from the V notch chamber should be let away in a channel, so that water pumped out does not find its way back into the well through the soil. As water from the well is pumped out, there will be stage where water level remains fairly constant, without any further increase in drawdown. The pumping rate in this position is the yield from the well for that head of depression or drawdown.

Aquifer Pumping Test of Wells

The pumping tests of wells enable determination of transmissivity (T) and storability (S) of aquifers which further help calculate decline in groundwater levels associated with pumpage data.

Methods of Solution:

Three methods, viz., (i) Theis, (ii) Copper and Jacob, and (iii) Chow are in use worldwide to obtain average values of T and S in the vicinity of a pumped well measuring effect of well pumpage (i.e., change in drawdown with time) in one or more observation wells under the influence of constant pumping rate.

Annexure 4.4: Disinfection of Wells and Tube Wells

Dug Wells

1. After the casing or lining is completed, the procedure outlined below may be carried out before the cover platform is placed over the well:
   (i) Remove all equipments and materials including tools, platforms, etc., which do not form a permanent part of the completed structure.
   (ii) Wash the interior walls of the casing or lining with a strong solution of the bleaching powder (50 mg/L chlorine) using a steel broom or brush to ensure thorough cleaning.
   (iii) Pump the water from the well until it is perfectly clear and remove the pumping equipment that was temporarily set up for this purpose.
2. Place the cover over the well and pour the required amount of bleaching powder solution in to the well through the manhole or pipe opening just prior to inserting the pump cylinder and drop pipe assembly. The bleaching powder added should give a dose of 50 mg/L of chlorine in the volume of water in the well. Care should be taken to distribute the chlorine solution over as much of the surface of the water as possible to obtain proper mixing of the chemical with well water, which may be facilitated by running the solution into the well through a hose or pipeline as the line is being alternatively lowered and raised.
3. Wash the exterior surface of the pump cylinder and drop pipe with bleaching powder solution giving 50 mg/L of chlorine when the assembly is being lowered into the well.
4. Allow the chlorine solution to remain in the well for not less than 24 hours.
5. After 24 hours or more have elapsed, the well should be flushed by pumping the water to waste, till the residual chlorine is brought to 1 mg/L.

Tube wells

1. When the well is tested for yield, the test pump should be operated until the well water is as clean and free from turbidity as possible.
2. After the testing equipment has been removed, pour the required amount of bleaching solution into the well slowly just prior to installing the permanent pumping equipment. The dose of chlorine should be maintained at 50 mg/L. Mixing of the chemical with well water may be facilitated by running the solution into the well through a hose or pipeline as the line is being alternatively raised and lowered.
3. Wash the exterior surface of the pump cylinder and drop pipe with bleaching powder solution before positioning.
4. Allow the chlorine solution to remain in the well for not less than 24 hours.
5. After 24 hours or more have elapsed, the well should be flushed by pumping the water to waste till a residual of 1 mg/L of chlorine is obtained. In the case of deep wells having a high-water level, it may be necessary to resort to special methods of introducing the disinfecting agent in to the well so as to ensure proper mixing of chlorine throughout the well.

Similar procedure is adopted when troubles due to iron bacteria are noticed in the tube wells particularly when they come out as stringy masses along with the water.

Disinfection of Pipelines

When a section of water main is laid or repaired, it is impossible to avoid contaminating the inner surface with dirt, mud, or water in the trench while the pipes are being fixed into place. Contamination may also occur by accident, negligence, or malice. Maintaining adequate surveillance during working hours and the plugging of open ends after the day's work will reduce these risks. It should be assumed, however, that the pipe is contaminated despite all the precautions taken to prevent the entry of foreign matter. Secondly the main must be disinfected before it is put into service.

To obtain good results from disinfection and to avoid the hazards of subsequent obstructions and damage to valves, all foreign objects and material should be removed beforehand by swabbing and flushing to clean the pipeline. Packing and jointing material should be cleaned and disinfected immediately before use by immersion in a 50 mg/L of chlorine solution for at least 30 minutes.

The presence of hydrants, air valves, gate valves and other openings in and around the section to be disinfected facilitate the injection and extraction of water for flushing and disinfection. Recently developed plastic foam swabs are also useful in disinfection of mains. As they are displaced by water pressure, these swabs wipe clean the inner surface of the pipe. They can isolate the section to be disinfected from the rest of the main and prevent the loss of disinfected solution.

Chlorine compounds are the most commonly used disinfectants for water mains. Strength of the disinfecting solution should be much higher than that normally used for water chlorination. Under normal conditions, a strength of 10 mg/L is recommended for a contact period of 12–24 hours. Application for 24 hours is necessary when the chlorine has to penetrate through organic matter coating the inner surface. In emergencies, when it is not possible to leave the section of the main out of service for a long time, the period of contact can be shortened by proportionately increasing the strength of the solution. Thus, for a contact period of one hour, the strength of solution varies between 120 mg/L and 240 mg/L. When strong solutions are used particular attention should be paid to thorough removal from the main after completion of disinfection as illness and discomfort may result from using highly chlorinated water and the corrosive action of the chlorine may damage pipes, valves, hydrants and household plumbing and fixtures.

Procedure for Application

Chlorine gas may be injected directly under the section of the main by a dry feed chlorine or supplied with a special gas diffuser or silver tube and attached to a hydrant or other opening by means of specially plugged valve. After the section has been thoroughly flushed, the entire valve is partly shut to bring water pressure below 1.70 Kg/cm2.

At the hydrant or opening where the water is discharged, the flow rate is measured to determine the rate at which chlorine gas needs to be delivered. To obtain a concentration of 10 mg/L in the section to be disinfected, the chlorine gas input rate should be 0.9 Kg per 24 hours for every litre per second of flow. The valve of the chlorine is opened and adjusted so that the dial shows the required rate of chlorine flow.

To ensure that the chlorine concentration remains at 10 mg/L throughout the period of contact, the strength of the injected solution should be at least twice as high. A table below shows the amount of disinfectants required for pipes of various diameters in order to provide a chlorine concentration of about 20 mg/L.

Quantity of disinfectants required to provide concentration of 20 mg/L in a 100 m pipe length.

Diameter of pipe mm	Quantity in litres in which disinfectant has to be dissolved 10 × Litre	Bleaching Powder (25% available chlorine) gm	Calcium Hypochlorite (70% available chlorine) gm	Sodium Hypochlorite (5% available chlorine) Litre
75	46	37	13	0.16
100	81	65	21	0.33
150	183	146	53	0.73
200	325	260	92	1.30
250	507	405	145	2.03
300	730	584	210	2.92
400	1,298	1,040	368	5.20

The volume in litres of the disinfecting solution required for 100 m of pipe can be expressed by $V= 0.08\ d^2$ where d is the diameter of the pie in mm.

As soon as the odour of chlorine is detected in water discharged from the main, water samples are taken to determine the chlorine content. When chlorine content reaches a value of 20 mg/L at the other end of the section being disinfected, the discharge hydrant is closed, and the flow of the water and chlorine gas are stopped. The water is allowed to stand in the main for 12–24 hours and the chlorine content should be ensured to be not less than 10 mg/L at the end of the period. The mains should be thoroughly flushed with treated water until the water is cleared. Samples for bacteriological tests should be taken every day during the three days following disinfection to ascertain that the water is satisfactory in quality.

A similar procedure is used for feeding a mixture of chlorine gas and water by means of a solution feed chlorinator. Special rubber hose should be fitted to the plug valve and the silver tube diffuser. A booster pump may be required to provide pressure at least three times higher than that in the main, in order to ensure satisfactory injection of the solution.

When calcium hypochlorite or chlorinated lime is used for disinfection of a section of a main, the easiest method of application is to inject a strong chlorine solution by means of portable chlorinator. If the intake valve is kept partly open, a small flow of water can enter the pipe to assist in the dispersion of the chemical. The discharge hydrant or valve is shut off when the odour of chlorine is detected in the water flowing out and the section of the main is allowed to fill. The intake valve is regulated so that the required amount of disinfecting solution is injected before the pipe is completely full.

When there is no chlorinator or pump to inject the disinfection solution, the intake valve is shut off after the flushing operation and the section is allowed to drain dry. Then the discharge hydrant or valve is shut off thus leaving the section to be disinfected, isolated from the rest of the main. The disinfecting solution is slowly poured through a funnel or a hose into an intermediate hydrant, valve or opening made for this purpose until the section is completely filled. Precaution should be taken to allow air trapped in the pie to escape; where there is no air valve or other orifice by which the air can be released, one or more service connections could be detached, or a hole could be drilled in the top of the pipe.

If the section to be disinfected is short, weighed quantities of calcium hypochlorite or chlorinated lime in powder form may be placed at regular intervals inside the pipes while they are fixed into place. When water is introduced later, the powder will mix with it and produce strong solution of chlorine. The disadvantage is that the powder will be flushed to the far end of the section even the water is admitted slowly, and no uniform distribution of disinfectant is possible.

While disinfecting solution remains in two pipes, the valves, and hydrants in the section of the main should be operated to ensure that all surfaces come into contact with disinfectant. The valves at either end of the treated section should remain shut during the whole period of contact to prevent the loss of disinfecting solution.

Annexure 5.1: Design Calculations for Pumping Machinery

1.0 Data of Scheme

1.1	Daily demand of water	116 ML/day
1.2	Hours of pumping, considering loss of one hour due to tripping and other minor interruptions	23 hrs per day
1.3.1	Maximum (Top water level)	11.0 m
1.3.2	Mean WL	9.0 m
1.3.3	Minimum WL	7.0 m
1.4	Rising main	
1.4.1	Length	2,575 m
1.4.2	Diameter (Internal)	1,200 mm
1.4.3	H-W Coefficient for MS mortar lined pipeline	140
1.5	RL of point of discharge destination	59.0 m
1.6	No. of pumps (Based on the preliminary analysis)	
	1.6.1 Duty pumps	4
	1.6.2 Standby pumps	2
1.7	RL of ground level at the pumping station	8.25 m
1.8	RL of high flood level	10.5 m
1.9	Altitude of the site above mean sea level	1,250 m
1.10	Ambient temperature	40 °C

2.0 Size of pipes and fittings for the pumping system for Q = 0.35 m^3/s per pump

2.1	Inlet bell mouth for VT pump	
	Design velocity	1.4 m/s
	Bell mouth diameter	0.564 m
		Say 600 mm
	Actual velocity	1.24 m/s

2.2 Column pipes

Design velocity	1.75 to 2.75 m/s
	Say, 2.75 m/s (for higher flow)
Column pipe diameter	0.402 m
	Say 400 mm

Actual velocity 2.79 m/s

2.3 Delivery pipes and valves

Design velocity	2.0 m/s
Diameter of delivery pipe, delivery valve and NRV	0.472 m
	Say 500 mm
Actual velocity	1.78 m/s

2.4 Suction pipes and valves

Design velocity	1.5 m/s
Diameter of suction pipe and valve	0.545 m
	Say 600 mm
Actual velocity	1.24 m/s

2.5 Suction bell mouth for centrifugal pump

Design velocity	1.2 m/s
Diameter of bell mouth	0.609 m
	Say 700 mm
Actual velocity	0.91 m/s

2.6 Suction eccentric reducer 600 × 500 (assumed)	1.78 m/s

3.0 Pump head and required head range

3.1 Combined discharge of four pumps In parallel (116 ML/d × 24 hrs.)/23 hrs.	121.01 MLD
3.2 Rate of total flow with 23 hrs. running of pumps per day	1.4 m^3/s

3.3 Discharge of each pump 0.35 m^3/s

3.4 Mean static head (59 m - 9 m) 50 m

3.5 Frictional loss in straight pipeline for combined discharge 2.237 m

3.6 Minor losses in bends, valves, and other fittings and exit loss in rising main at 10% of (3.5) 0.2237 m

3.7 Station loss (head loss in suction, delivery piping, valves, manifold, etc., at pump house).

Table Showing head losses in piping and valves in the pumping station

S. No.	Fitting/Equipment	Size (mm)	Velocity (m/s)	K/C*	Head loss
A) Delivery side (VT/Centrifugal Pump)					
1	Enlarger	400 × 500	2.79	0.4	0.16
2	Tee for air valve	500 × 500 × 100	1.78	0.3	0.05
3	Non-return valve	500	1.78	2.5	0.40
4	Dismantling joint	500	1.78	0.3	0.05
5	Delivery valve (BFV)	500	1.78	0.4	0.06
6	Distance piece 1 m length	500	1.78	110	Negligible
7	Knife gate valve	500	1.78	0.3	0.05
8	30°/45° tee at manifold	500	1.78	0.8	0.13
9	Manifold (L = 14 m)	1,200	1.24	110	0.02
10	90° bend	1,200	1.24	0.75	0.06
11	NRV/DPCV	1,200	1.24	2.5	0.19
12	Dismantling joint/flange adopter	1,200	1.24	0.3	0.02
13	Isolation valve (BFV)	1,200	1.24	0.4	0.03
14	Flowmeter	1,200	1.24	0.1	0.007
				Total (A)	1.23 (rounded 1.3)
B) Suction side (centrifugal pump only)					
15	Bell mouth	700	0.91	0.1	0.004
16	90° bend	600	1.24	0.5	0.04
17	BFV	600	1.24	0.4	0.03

S. No.	Fitting/Equipment	Size (mm)	Velocity (m/s)	K/C*	Head loss
18	Suction piping/reducer	500 × 600	1.78	0.20	0.20 (assumed)
				Total (B)	0.28 (rounded 0.3)

*Head loss in valves and fittings = k x $\frac{v^2}{2g}$

Where k = Coefficient depending on valve/fitting

(Refer Chapter 6 Part A of Manual)

*H_f in pipes = $\frac{10.674}{D^{4.87}} \times \left(\frac{Q^{1.852}}{C}\right) \times L$

Where H_f = friction loss in straight pipeline

Q = discharge, m^3/s

L = Length of pipe

D = internal diameter, m

3.8 Design pump head for VT pump = (3.4) + (3.5) + (3.6) + Total A = 54.0 m (Rounded)

3.9 Design pump head for centrifugal pump = (3.4) + (3.5) + (3.6) + Total A + Total B

= 54.3 m (Rounded)

3.10 System head curve

Table showing co-ordinates of heads and discharge for system head curves at max WL, mean WL, and min WL (discharging RL at destination = 59.0 m).

Parameter		H_f	Total H at max WL (m)	Total H at mean WL (m)	Total H at min WL (m)
WL	-	-	11.0	9.0	7.0
Static head	-	-	48.0	50.0	52.0
Q(m^3/s)	0	-	48.0	50.0	52.0
	0.25	0.101	48.101	50.101	52.101
	0.50	0.365	48.365	50.365	52.365
	0.75	0.775	48.775	50.775	52.775
	1.00	1.314	49.314	51.314	53.314
	1.25	1.986	49.986	51.986	53.986
	1.50	2.783	50.783	52.783	54.783
	1.75	3.703	51.703	53.703	55.703

The system head curve is illustrated in Figure 5.1.A.1.

3.11 Head Range

As seen from the system head curve, actual head variations are from 48.0 m to 55.7 m which is -11.1% to +3.1%. However, as per guidelines and IS, the head range for the VT pump (which is finally selected) shall be +10% and -25%.

Hence, head range based on duty head of 54.0 m shall be 59.4 m to 40.5 m.

4.0 Selection of Type of Pumps - VT or Horizontal Centrifugal Pumps

4.1 VT Pump

VT pump shall be installed at a floor level above the maximum WL in the sump. The maximum WL is 11.0 m, which is above HFL at 10.5 m. There is no necessity for priming in case of VT pumps. Hence, a VT pump is feasible and generally suitable for any installation with sump and pump house located above sump.

4.2 Horizontal Centrifugal Pump

The pump shall preferably be a double suction, single stage, horizontal split casing so as to have a low NPSHr.

It is necessary to check the feasibility of a centrifugal pump installation to ensure that the pump mounting floor is not dangerously below GL and the risk of the motor getting waterlogged if any valve or piping inside the pump house burst. The feasibility checks shall cover the following two vital points.

i) The top of volute of the pump should be below minimum WL by the magnitude of head losses in suction piping and valve to ensure that the pump can be fully primed without any vacuum pump
ii) Suction specific speed (Nsss) of the pump at the site installation shall be below 145 (SI)/7,500 USCU as discussed in Chapter 5, subsection 5.8.4.

4.2.1 Pump installation level from the aspect of the case of priming

Size of pump volute and installation level
Figure 17.40 and the equation for impeller diameter in the book ‘Centrifugal Pump’ by Karassik can be referred to calculate impeller diameters. See Figure 5.1.A.2 reproduced. The dimensions are head in feet and Q in US gpm.

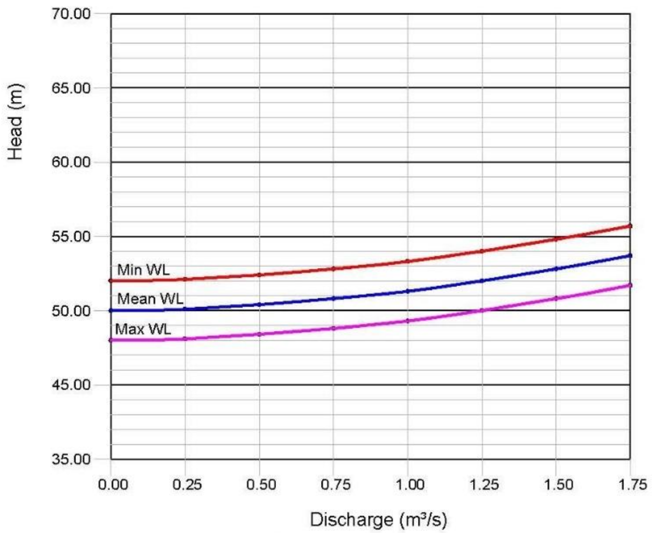


FIGURE 5.1.A.1 : SYSTEM HEAD CURVES

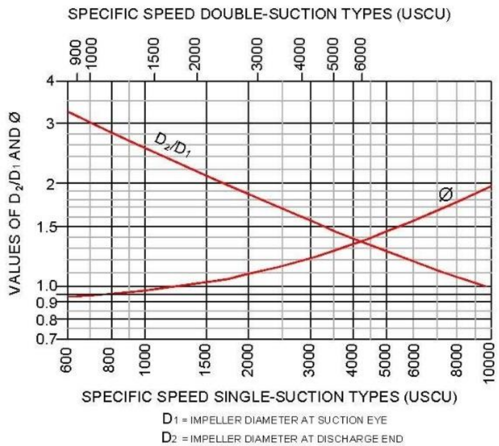
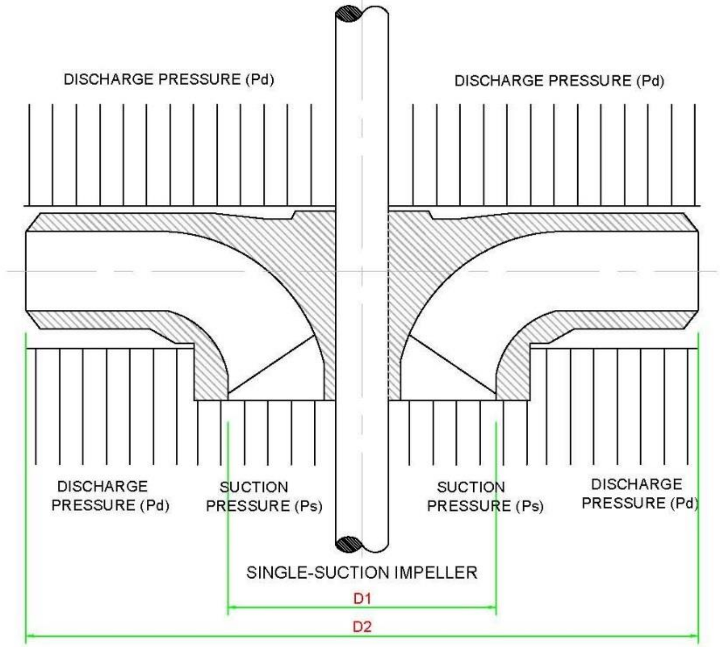


FIGURE 5.1.A.2 : VARIATION OF D_2/D_1 RATIO AND ∅ WITH SPECIFIC SPEED



A_{unbl}, Unballanced Area for Hydraulic Thrust = Pi/4 x D_1^2

P_{nett} = Pd - Ps

Hydraulic Thrust = 0.8 x A_{unbl} x (Pd - Ps)

Total Axial Thrust = Hydraulic Thrust + Weight of Line Shaft, Impeller & Impeller Shaft

FIGURE 5.1.A.3 : ILLUSTRATION AND MAGNITUDE OF HYDRAULIC THRUST

Q = 0.35 m^3/s = 5551 US gpm
H = 54.0 m = 177.17 feet

Ns (specific speed) = $N\sqrt{Q}\ /H^{0.75}$

Where N = rpm
Q = US gpm
H = head, feet

If N = 980 rpm, Ns = 1,503 USCU

and N = 1,480 rpm, Ns = 2,271 USCU

Referring to subsection 5.7.1 of the Chapter Ns = 1,503 USCU is low and not suitable. Ns = 2,271 USCU is acceptable. Hence, the pump speed shall be about 1,480 rpm (1,500 rpm synchronous) if a centrifugal pump is selected.

From the Figure 5.1.A.2 for Ns = 2271 USCU (double suction pump)

ϕ = 1.03

Using the equation given in the book,

H = $\left(\frac{D_2 N}{1840\phi}\right)^2$ …… H in feet and diameter of impeller, D_2 in inches

Substituting and on calculation

D_2 = 17.05 inches = 433 mm

Usually, volute outside diameter is 2 to 2.25 times of impeller diameter

Volute outside diameter = 975 mm

Assuming that the combined height of feet of pumps, base plate, and c.c. foundation as 500 mm,

Minimum height of the top of volute above pump floor level

= 975 + 500 = 1475 mm = 1.475 m = 1.5 m

The safe margin for assumptions and variation of the volute size of different pump manufacturers

= 200 mm = 0.2 m

Therefore, the design height of the top of volute above the pump floor

= 1.5 + 0.2 = 1.7 m

Head loss in suction piping, fitting, and valve (as per 3.7 above) = 0.3 m = 300 mm

Hence pump floor level = Min WL - 1.7 - 0.3

= 7.0 - 2.0

= 5.0 m RL

4.2.3 Pump Installation level from the aspect of safe suction specific speed (Nsss)

For cavitation-free operation,

Nsss ≤ 145 (SI)

Using the equation in subsection 5.8.4, Nsss = $\frac{N\sqrt{Q}}{NPSHa^{0.75}}$

Where, NPSHa = Net positive suction head available

Here for Nsss, half Q for double suction pump is to be considered. Solving,

NPSHa ≥ 6.92 m

NPSHa at site installation (refer to subsection 5.8.3)

= Ps - Hfs - $\frac{Vs^2}{2g} - Zs - V_p$

(The parameters are defined in the relevant subsection, hence not repeated).

Here Ps = 9.55 m (atmospheric pressure at an altitude of 1250 m above MSL)

Hfs = 0.3 m (as before)

$\frac{Vs^2}{2g} = \frac{1.78^2}{2xg} =$ 0.16 m (assuming 500 mm suction nozzle for 600ϕ suction pipe)

Vp = 0.427 m (at 30°C water temperature for 40°C ambient)

Hence,

Zs ≤ 9.55 - 0.3 - 0.16 - 0.427 - 6.92

≤ 1.74 m, say 1.75 m

+ve value of Zs implies the suitability of the pump for a 1.75 m suction lift.

Hence, RL at the eye of the impeller ≤ Min WL + Zs

≤ 7 + 1.75

≤ 8.75 m RL

Based on values calculated in 5.2.2 above,

Pump floor level ≤ 8.75 - $\frac{900}{2x1000} - \frac{500}{1000}$

$\leq$ 7.80 m RL

4.2.4 Conclusion about pump installation and recommendation

As seen from 4.2.2 and 4.2.3, the lowest pump floor levels worked out are:

i) For priming 5.0 m RL
ii) For Nsss criteria 7.80 m RL, which necessitates a vacuum pump for priming and complex automation

Thus, design pump floor level shall be 5.0 m RL, i.e., lower of these two values. Ground level at site = 8.25 m RL (as per data).

It is seen that the pump floor level shall be 8.25 - 5.0, i.e., 3.25 m below GL.

In the event of any burst in delivery piping or valve, water will accumulate in the pump house up to at least 3.25 m causing submergence of motors necessitating motor rewinding or insulation, and other major repairs. Such mishappenings have taken place in some badly designed centrifugal pump installations.

Therefore, the selection of a centrifugal pump is with serious risk and not advisable.

5.0 Selection of number of stages and rotating speed (rpm) of VT Pump

Pump head as per 3.9, H = 54.0 m

Head loss at the entrance to 600 mm diameter bell mouth

$H_e = 0.1\ V^2/2g = 0.007$ m

Head loss in column pipe, assuming presently, 10 m length of the column and as per Figure (5) in IS 1710-1972 for 400 mm diameter,

$H_c = 0.25$ m

H_d, Head loss in discharge bend/tee = 0.20 m

(Note: k = 0.5, v = 2.79 m/s, head loss = $kV^2/2g$)

Hence, bowl head, $H = 54.0 + H_e + H_c + H_d = 54.457$ m, Say 54.5 m.

Six options on the basis of variation in the number of stages from 1 to 3 and speeds 980 and 1,480 rpm as per Table below, are evaluated.

Table showing options and evaluation of Ns and efficiency for variations in the number of stages and speed [$Q_{(pump)} = 0.35\ m^3/s$ and $H_{bowl} = 54.5$ m].

Option	Number of Stages	Head per stage (m)	rpm	Specific Speed (SI)	Graph efficiency	Remarks
1	1	54.5	980	28.9	0.86	Not accepted, unstable H-Q curve

2	1	54.5	1480	43.65	0.90	Acceptable
3	2	27.25	980	48.61	0.90	Acceptable
4	2	27.25	1480	73.41	0.87	Acceptable
5	3	18.17	980	65.87	0.88	Acceptable
6	3	18.17	1480	99.49	0.85	Not acceptable: shut-off kW is higher than kW at BEP

It is seen from the above analysis that options 2 and 3 are to be compared judiciously for final selection.

Considering that wear and tear and noise level at 1,480 rpm shall be more than that of 980 rpm, the most appropriate option shall be Option 3 (980 rpm two-stage VT pump).

6.0 Sizes/Rating of other design components and parameters

6.1 Motor

i) Lowest bowl efficiency as per 4 above is 0.90

Allowing 5% variation for commercial offer design bowl efficiency is 0.85

ii) Input to bowl assembly (refer to subsection 5.11)

= 9.81 × 0.35 × 54.5 / 0.85

= 220.15 kW

iii) Thrust bearing loss (kW) = $\frac{0.0009225 \, X \, T \, X \, N}{100 \, X \, 50}$ = 1.08 kW

Where

T = thrust in kg = F = 5,000 kg (refer to 6.2 below)

N = operating speed rev/min = 980 rpm

iv) Line shaft bearing loss = 0.5 kW (as per IS 1710)

v) Input to pump [(ii) + (iii) + (iv)] 221.73 kW

Considering 10% margin of power in motor, rating of motor required 243.90 kW

Rounding to commercial rating 250 kW

Note: Calculations for motor rating are to be done to enable detailing specifications for associated electrical equipment. Motor rating should not be specified in the tender specifications.

vi) As the rating is 250 kW, as seen from subsection 5.20.5, HT motors at 3.3 kV can be adopted. In case of a generally clean, dry environment, Screen Protected Drip Proof

(SPDP) or Open Drip Proof (ODP) motors (IP23) can be selected as they allow air to circulate freely through the windings for cooling and prevent water dripping from above entering the windings. This is normally adequate for most of the indoor pump sets. In case of dusty environment, frequent cleaning of the vents, (and fly and moth menace during nights which may be sucked into the motor) and risk of water splashes, select totally enclosed, fan cooled, IP44 (TEFC) motors which allow external air inside the motor through a fan, but are safe against not only water dripping from above, but also water splash.

vii) The motor shall be designed for Class F insulation with a temperature rise limit for Class B. As the altitude at the site is 1250 m RL, the temperature rise limit shall be reduced by 2.5 °C, i.e., at the rate of 1 °C for every 100 m higher altitude above 1,000 m RL.

6.2 Impeller shaft and line shaft diameter

i) IS 1710: Specification for vertical turbine pump stipulates the following formula for impeller shaft. The same can also be applicable to the line shaft.

$$S = \sqrt{\left(\frac{63.7F}{D^2}\right)^2 + \left(\frac{492p \times 10^6}{ND^3}\right)^2}$$

Where S = combined shear stress in kgf/cm^2 shall be either a maximum 30 per cent of the yield stress or maximum 18 per cent of the ultimate tensile strength of the shaft steel used

F = axial thrust in kg of the shaft, including hydraulic thrust plus the weight of the shaft and all rotating parts line shaft, impeller shaft, impeller, etc., supported by it

p = power transmitted by the shaft in kW (advisable to adopt motor kW)

N = revolutions per minute (rpm)

D = shear diameter in mm at the root of threads or minimum diameter at undercut

ii) Calculations for hydraulic thrust

The first step is to determine, impeller diameters - outside diameter D_2 and inside diameter D_1.

Ns (specific speed) = $\frac{N\sqrt{Q}}{h^{0.75}}$

Where N = rpm

Q = discharge, m^3/s

h = head per stage, m

h = $\frac{54.5}{2}$ = 27.25 m, 89.41 feet

N = 980 rpm

Q = 0.35 m^3/s, 5551 USgpm

Hence, Ns = 48.61 (SI), 2512.5 (USCU)

From Figure 17.40 from the book Centrifugal Pump by Karassik, see reproduced Figure 5.1.A.2.

D_2/D_1 = 1.55 and ϕ 1.15

Using the equation from the above book,

$\phi = \frac{D_2 N}{1840\sqrt{h}}$

Substituting for ϕ = 1.15, h = 89.41, N = 980 and solving,

D_2 = 20.416 inches = 0.518 m

Hence, D_1 = 0.335 m (Nominal)

Adding 2×6 mm (approx.) thickness of wearing ring, D_1 = 0.347 m

As per guidelines on Page 2.276 and 2.277 of Pump Handbook, 4th edition, refer to Figure 5.1.A.3 for an illustration of hydraulic thrust.

Unbalanced area = $\frac{\pi}{4} x D_1^2$

= 0.095 m^2 = 946 cm^2

Unbalanced pressure (neglecting and assuming zero pressure on the suction side of the eye of the impeller due to submergence being of very low magnitude).

= 54.5 m = 5.45 kg/cm^2

As per guidelines in the Pump Handbook, 70%-80% thrust is to be considered

Hence hydraulic thrust = 946 × 5.45 × 0.8

= 4125 kg

iii) Weight of 10 m long 65 mm diameter line shaft (and density 7,850 kg/m^3)

= $\frac{\pi\, x\, 0.065^2 x 10 x 7850}{4}$

= 261 kg say 300 kg

iv) Weight of impeller (assuming void factor as 0.5 and width as 33% of D_2)

= $\frac{\pi\, x\, D_2^2 x 0.33 D_2 x 7850 x 0.5}{4}$

= 141.4 kg say 200 kg

v) Weight of Impeller shaft = 50 kg (assumed)

Hence axial thrust, F = (ii) + (iii) + (iv) + (v)

= 4125 + 300 + 200 + 50

= 4475 kg, say 5,000 kg

SS 410/416 as MOC of impeller shaft and line shaft is selected

Yield stress = 28.2 kg/mm^2 = 2820 kg/cm^2

Ultimate strength = 49.3 kg/mm^2 = 4930 kg/cm^2

Hence safe stress S = 30% of 2820 kg/cm^2 or 18% of 4930 kg/cm^2 (whichever is lower)

= 846 kg/cm^2 or 887 kg/cm^2 (lower of two)

= 846 kg/cm^2

Substituting values in the equation, (F = 5,000, p = 250, N = 980, S = 992) and solving by trial and error (as the equation is not linear)

D = 53 mm at the root of screw threads or undercut due to sleeve

Adding 2 × 5 mm depth of screw threads and 4 mm corrosion allowance and tolerances

Nominal shaft diameter = 67 mm

6.3 Thickness of Column pipe

The column pipe will act as a closed pressure vessel when the pump is started under shut-off conditions. Considering the specific speed and pattern of pump characteristics, the shut-off head is likely to be 80 m.

The material of construction shall be mild steel.

Hence design pressure (at 1.5 times shut-off pressure)

P = (80/10) × 1.5 = 12 kgf/cm^2

For pressure vessel as per IS 2825-1969

$t = (PD)/(200f_i\text{-}p)$

Where p, design pressure = 12 kgf/cm^2

D, internal diameter = 400 mm

F, safe stress = 10kgf/mm^2

J, welding factor = 0.7

Hence, t = 3.45 mm

Adding 4 mm corrosion allowance as the pipe is subjected to corrosion from both inside and outside.

Thickness of column pipe = 7.5 mm = 8mm

Design rating of electrical equipment

7.1 Transformer

a) Total load of four pump motor sets 250×4 = 1,000 kW

Hence transformers kVA required at 0.85 P.F. and 20% margin

$= \frac{1000\times1.2}{0.85}$ = 1412 kVA

The next commercial ratings are 1,600 kVA and 2,000 kVA. in addition to a load of main pump motors, load due to ventilation system, indoor and external lighting, power for crane, surge control devices, and water treatment plant are necessary. Momentary starting overload of transformer on starting last working pump should not exceed 50% of transformer capacity.

Considering these aspects, a transformer of 2000 kVA, one duty + one standby is selected.

b) Check for loading under the last/4th motor starting

Momentary overloading of the transformer to be checked for condition N-1 number of pumps running and Nth pump started.

Here Number of duty pumps = 4

kW each motor = 250 kW

Hence , kVA load for three pump motors running (assuming worst PF 0.85 and motor efficiency = 0.90)

$= \frac{250\times3}{0.85\times0.9}$ = 980.4 say 981 kVA

Maximum starting kVA of 4th pump as six times being direct online starting

$= \frac{250\times6}{0.85\times0.9}$ = 1,960.8 kVA, Say 1,961 kVA

Hence kVA load when 4th pump started

= 981 + 1,961 = 2,942 kVA

Momentary overload = $\frac{2{,}942}{2{,}000}$ - 1

= 47% (within the limit of 50%)

Hence acceptable

7.2 Breaker on incoming to transformer

The power supply authority's system is 22 kV (Design 24 kV) and the fault level is 500 MVA.

Therefore, S.C. current $\frac{500}{1.732\times24} = 12.03\ kA$

Full load current of transformer $\frac{2000}{\sqrt{3}\times24}$ = 48.11A

Standard rating - 24 kV, 12.5 kA, 6,30A

7.3 Motor Control Gear

As the motor is 3.3 kV, vacuum circuit breaker (VCB) can be selected.

(i) Current at 0.85 P.F. and lowest voltage 3.3 kV - 10%, i.e., 2.97 kV

$= \frac{250}{\sqrt{3} \times 2.97 \times 0.85 \times 0.9} = 63.53A\ (0.9\ motor\ efficiency)$

Specify minimum available rating is 400A as per IS 13118.

(ii) Short Circuit Current rating

Normal impedance with 10% tolerance as per IS 2026 = 6.25 × 0.9 = 5.6%

Z_{min} = 5.6%

Therefore, short circuit MVA of a transformer at 3.3 kV Bus

= (2000 × 100)/(5.6 × 1000) = 35.71 MVA

As motor contributes 10 times its normal full load during fault, contribution of four motors.

S.C. Current = 63.53 × 10 × 4 = 2.54 kA

S.C MVA = 3.3 × 2.54 × $\sqrt{3}$ = 14.52 MVA

Hence, total S.C. MVA = 50.23 MVA

Hence, short circuit breaking capacity of the breaker at 3.6 kV design voltage as per IS 13118

$= \frac{50.23}{\sqrt{3} \times 3.6}$ = 8.05 kA

Breaker rating: 3.6 kV, 400A, 10 kA as per IS 13118

7.4 Incoming breaker and bus-coupler breaker to HT Panel

Normal current = 63.53 × 4 = 254.12A

Specify 630A as per IS 13118. Considering margin for future.

Breaker rating: 3.6 kV, 630 A, 16 kA as per IS 13118

S.C. MVA shall thus be 99.8 MVA

Hydraulic design of Sump and General Arrangement of Pump House

8.1 Sump

As per guidelines in subsection 5.2.10.2,

a) Clearance between the bottom of sump and lip of inlet bell mouth,

C = D/2 = 600/2 = 300 mm

b) Submergence above bell mouth and water depth

Diameter of inlet bell mouth	0.6 m		
V (actual) [Refer 2.1]	1.24 m/s		
F_D, Froude number	$\frac{v}{\sqrt{g}}$	=	0.511

S, minimum submergence required above Lip of bell mouth

= $D \times [1+2.3\ F_D]$

= 1.30 m

C (calculated as before) = 0.3 m

$\therefore$ Minimum depth of water below minimum WL (S+C)

= 1.60 m

c) Distance between rear wall and centre of bell mouth,

B_{max} = 0.75 D = 0.75 × 600 = 450 (maximum) = 450 mm

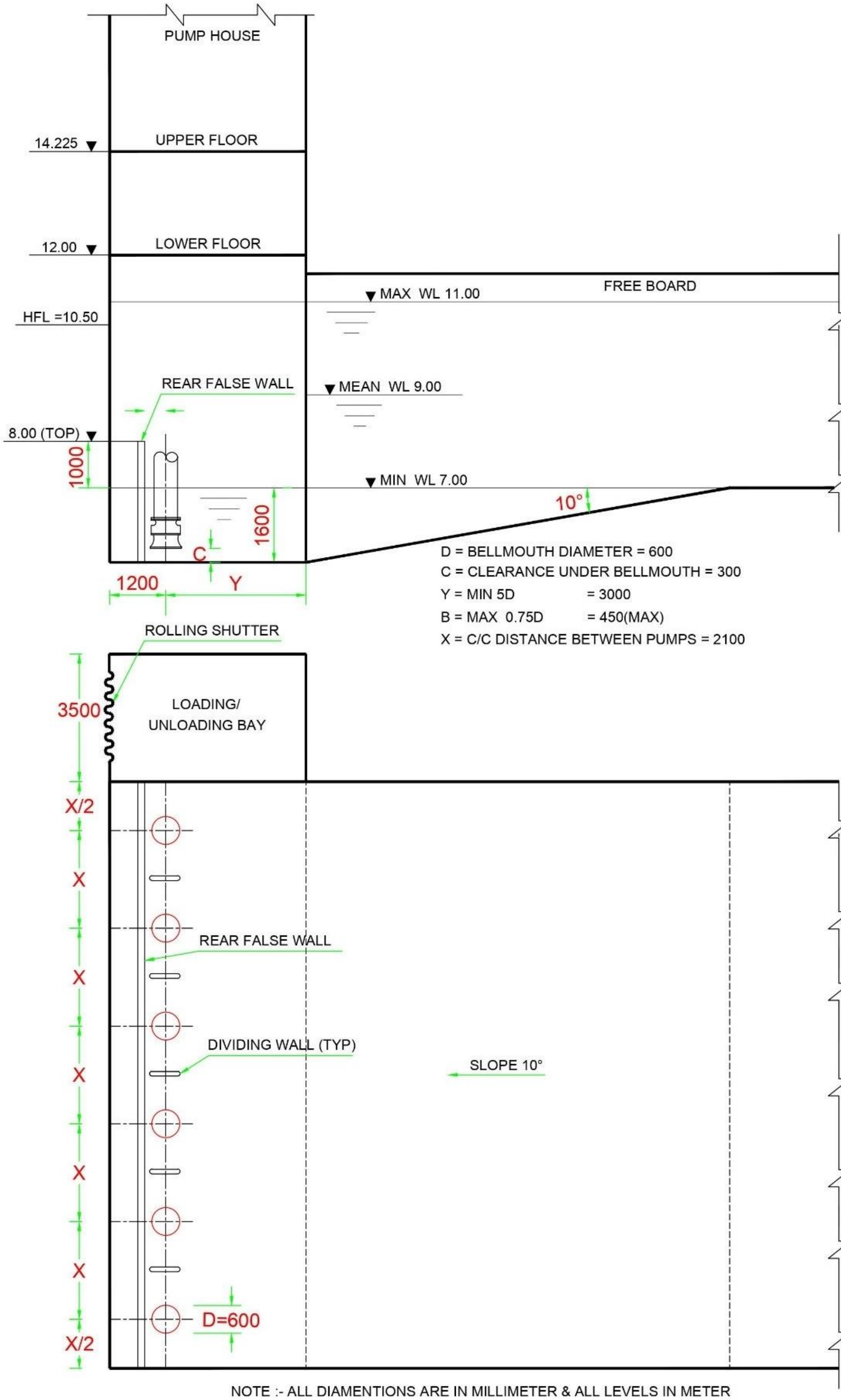


FIGURE 5.1.A.4 : DIMENSIONS OF SUMP FOR V.T. PUMPS FOR VORTEX-FREE OPERATION

d) Rear False Wall

The size of the base of the discharge head will be 1200 mm, i.e., 600 mm from the centre of the pump, whereas dimension B is 450 mm (max). Therefore, the column and rear wall of the sump will have to be located at least 1,200 mm away from the pump centre keeping a 600 mm margin for nut fastening, etc. Therefore, a rear false wall is necessary at a distance of 450 mm (clear) from the pump centre. A top or false wall will be up to 1000 mm above the maximum water level.

e) Spacing between pumps

Desirable spacing between pumps is 2.5 D, i.e., 1,500 mm. However, the size of the lower flange of headgear/discharge head (accommodating stuffing thrust bearing and flexible coupling would be approximately 3.0 times column pipe diameter, i.e., 1,200 mm. Keeping about 900 mm clearance, the spacing will be 2,100 mm.

f) Slope

As the seen minimum depth of water required is 1.60 m below the minimum WL. In order to minimise excavation cost, a permissible slope of 10° degree is taken. The slope will terminate upstream of the pump at a distance equal to 5 D, i.e., 3,000 mm from the pump centre.

g) Straight Approach

The portion under the pump will be flat from the line of termination of slope up to at least the rear false wall.

h) Baffles/Dividing Walls

Dividing walls will be constructed between pumps to avoid mutual interference. Both ends of each dividing wall shall be rounded. The front edge of the dividing wall shall be in line with the front edge of the suction bell mouth. At the rear end opening 150-200 mm size shall be kept at least up to minimum WL. The top of the dividing wall will be up to 1,000 mm above the minimum WL.

8.2 Pump House

a) Number of floors

As the delivery size is 500 mm, two floors shall be planned. The lower floor for pump installation and delivery piping and valve and the upper floor shall be the operating floor. The lower floor shall be 1.25 m above HFL of 10.5 m or keeping 0.6 m freeboard above max WL of 11.0 m, which works out 11.75 m RL, say 12.0 m.

Headroom between floors shall be 2250 mm.

b) Corbel level for EOT crane shall be 5.5 m above the upper floor
c) Headroom above corbel shall be minimum of 2.0 m for the bridge girder of the crane.

Capacity of EOT Crane

9.1 Design Basis

EOT crane should have a minimum 25% higher capacity to lift maximum load in the pumping station. The loads to be lifted are as follows:

i) The motor can be de-flanged from the discharge head and lifted as separate equipment.

ii) For dismantling column pipe or bowl assembly, the load to be lifted as a single load comprises the following:

- Discharge head
- Column assembly
- Bowl assembly
- Inlet bell mouth

Load (ii) shall be higher than load (i)

9.2 Load (ii) - Discharge head, column assembly, etc.

a) Weight of discharge head

The base of the discharge head shall generally be about 2.5 to 3.5 times column pipe diameter and height shall be about 1.25 times base and solid-void factor = 0.25. The density is 7850 kg/m^3.

Weight $= 0.7854 \times (0.4 \times 3.5)^2 \times 1.25 \times 0.25 \times 7850$

$= 3776$ kg

b) Column assembly

Diameter = 0.4 m

Thickness = 8 mm = 0.008 m

Length = 10 m

The factor for the weight of flanges = 20%

line shaft, etc.

$\therefore$ Weight $= \pi \times D \times t \times L \times 7{,}850 \times 1.2$

$= 947.0$ kg

c) Bowl assembly

Impeller diameter (D_2) worked out above

= 0.518 m

Bowl diameter = 1.15% of impeller diameter

= 0.6 m

Height of bowl = 1.25 times bowl diameter

= 0.75 m

Solid-void factor = 0.5

Weight = $\frac{\pi}{4} \times 0.4^2 \times 0.75 \times 0.5 \times 7{,}850$

= 370 kg

d) Inlet bell mouth (VT Pump)

Weight	=	150 kg (assumed)
∴Total load	=	(a) + (b) + (c) + (d)
	=	5,243 kg
	=	Say 5,500
Capacity of crane	=	1.25 × 5,500
	=	7,000 kg

Dynamic load of pump-motor set on Civil structure

10.1 Dead Loads

a) Due to the pump set

As worked out in 9.0 above the weight of pump set = 5,500 kg

b) Motor

As per enquiry,
Weight of motor = 3,000 kg

c) Weight of base frame and sole plate

= 500 kg (assumed)

d) Weight of water in column assembly

Volume of water 10 m long column assembly

= $\frac{\pi}{4} \times 0.4^2 \times 10 \times 1{,}000$
= 1,260 kg

10.2 Dynamic load for the design of civil structural members

As per IS 2974 Part IV, and as discussed in subsection 5.15(b) of Chapter 5 Part A, a dynamic factor of 3 is applicable for pumps and 2 for the motor. This dynamic factor is logical for a centrifugal pump mounted on a foundation. In the case of the VT pump, the bowl assembly is submerged in water and hence vibrations are dampened. Vibration in discharge head and column assembly are of low magnitude.

Considering the above, low dynamic factor 2 instead 3 is applied for dead weight bowl assembly, column assembly, and discharge head. For water in column assembly, a dynamic factor of 1.25 is considered. For motor, the factor shall be 2 as per IS 2974.

Hence, dynamic load = 5,500 × 2 + 3,000 × 2

+ 500 × 2 + 1,260 × 1.25

= 19,575 kg

Adding 10% for inaccuracy in estimation,

Design dynamic load on civil structure = 19,575 × 1.1 = 21,532 kg

= 22,000 kg (rounded)

Annexure 5.2: CFD Analysis

CFD analysis for vortex-free hydraulic design of intake and sump is developed over last two decades as acceptable alternative to sump model test which is very expensive and time-consuming. Originally CFD analysis was evolved for pump impeller, volute, suction and delivery nozzles, and other components through which flow passes. The basis for all CFD analysis is the Navier-Stokes equation and Reynolds number. The CFD analysis has partly replaced conventional methodology making sump models and testing.

The CFD analysis evolved for pumps is further applied for hydraulic design of intake and sump. The outcomes are highly dependable. Recently numbers of commercial software are available. Any modification or corrective measures required for prevention of vertices can be analysed and concluded.

However, the literature recommends that measures determined by CFD analysis shall be reconfirmed by conducting sump model test.

1.0 Sump Model Test

Basis for sump model test is Froude number which is non-dimensional parameter and kept same for prototype and model. The Froude number F_D is given by,

$$F_D = \frac{V}{\sqrt{gL}}$$

Where V = Flow velocity

g = acceleration due to gravity

L = any dimension of prototype/model

If R = Linear scale ratio of model to prototype

L_m = Any linear dimension of model

L_p = Corresponding linear dimension of prototype

Q_m = Discharge in model

Q_p = Discharge in prototype

Then $\mathrm{R} = \frac{\mathrm{Lm}}{\mathrm{Lp}}$

$$\mathrm{Vm} = \mathrm{Vp} \times \sqrt{\mathrm{R}}$$

$$\mathrm{Qm} = \mathrm{Qp} \times \mathrm{R}^{2.5}$$

Thus, if Vp = 1.2 m/s, Qp = 2.4 m^3/s and ratio R = $\frac{1}{4}$

Then, $V_m = 1.2 \times \sqrt{\frac{1}{4}}$ = 0.6 m/s

$Q_m = 2.4 \times (1/4)^{2.5}$ = 0.075 m^3/s

Essential requirements for sump model test

(i) In model, all physical dimensions such as width of pump bay, approach channel, slope, bottom clearance, back clearance, centre to centre distance between bell mouths/suction inlets should be in the scale ratio.

(ii) Submergence is linear dimension. Hence, submergence measured from lip of bell mouths/inlet to minimum water level shall be in ratio R. In the example, if submergence for prototype is 1.9 m, the submergence for model shall be $1.9 \times \frac{1}{4}$ = 0.475 m.

The test should be conducted at minimum required submergence for model.

(iii) The external surfaces of the bowl assembly including bell mouth, and internal geometry from bell mouth to eye of impeller must be modelled to scale R.

(iv) Model test should be conducted at two velocity values, i.e., velocity as per Froude number and also at least 1.5 to 2 times the Froude velocity.

(v) Any surface swirl, surface dimple, dye core air bubble, or air core should be observed at free surface and subsurface (below water level)

Annexure 6.1: Economical Size of Pumping Main

In view of the principles of design given in the subsection for the economic size of the pumping main, a stepwise design process is given below in the form of an excel sheet in print media which can be easily written in electronic media:

1. Establish data required for pumping main.
    a. Design period - 30 years
    b. Design life of pump sets - 15 years
    c. Supply of water in two stages each of 15 years
    d. First stage supply rate is the average of the initial and intermediate flow rate in MLD (for energy cost only)
    e. Second stage supply rate is the average of intermediate and ultimate flow rate in MLD (for energy cost only)
    f. Hours of pumping - 22 hr
    g. The cost of pipes, at prevailing rates in Rs/metre (if required, the cost includes the cost of excavation, lowering, laying, and jointing of pipes and specials filling of trenches, etc.) can be included. However, these costs are, more or less, constant in different alternatives.
    h. Cost of first stage pump sets at prevailing rates in Rs/each pump set (pump+ electric motor)
    i. Similarly, obtain the cost of second stage pump sets
    j. Cost of electrical energy at prevailing rates in Rs/kWh
    k. Cost of operation and maintenance cost as per estimate framed for pumping main in Rs/year
    l. Average hour of pumping (1^{st} Stage) = (S. No. (f)/intermediate flow rate) × average flow rate for first stage
    m. Average hour of pumping (2^{nd} Stage) = (S. No. (f)/ultimate flow rate) × average flow rate for second stage
2. Select appropriate pipe material considering topography, soil condition, as well as the quality of water to be conveyed.
3. Select different sizes of pipes of chosen material by using Lea's formula.
4. Calculate velocity and frictional losses in pipe length of pumping main and appurtenances for first stage intermediate flow rate and for second stage ultimate flow rate for the selected pipe size.
5. Determine the class of pipe considering total pressure head (static head, i.e., difference of level between the axis of pump and delivery point + head losses due to friction in pipe length of pumping main and its appurtenances for ultimate flow + residual head required at the delivery end). Also consider surge pressure, calculated by formula as given in the section of 'WATER HAMMER'.

6. Determine the total head of pumping of the first stage (static head + frictional losses in step 4 + residual head) and for the second stage (static head +frictional losses as given in step 4 + residual head).
7. Determine kw of pump set taking the total head of pumping and flow rates as given in steps 4 and 6. Add 50% for standby pump sets. (Calculate pump kw by the formula given in "Chapter 4 Pumping Station and Machinery".
8. Calculate the annual cost of energy as follows:

   The annual cost of energy for the first stage = kW of pump × average hours of pumping/day (as given in step 1(l) × 365 days × cost of electrical energy at the prevailing rate as in 1(j).

   Similarly, calculate the annual cost for the second stage taking average pumping hours as accurately given in step 1(m).
9. Capitalise annual cost of energy and O&M for first and second stage as given:

   First Stage capitalised energy cost = $AEC\left\{\frac{1-(1+r)^{-n}}{r}\right\}$ (Eq. 1)

   Where:

   AEC = Annual Energy Cost

   r = Rate of interest

   n = Number of years

   First stage capitalised O&M cost = $AOM\left\{\frac{1-(1+r)^{-n}}{r}\right\}$ (Eq. 2)

   Where A.O.M = annual operation/maintenance cost.

   Similarly, calculate the capitalised cost for energy and O&M for the second stage.
10. Calculate the PW of future capitalised cost of energy and O & M of the second stage.

$$\text{Present worth second stage energy cost} = \frac{\text{Capitalized 2nd stage energy cost}}{(1+r)^n} \quad \text{(Eq. 3)}$$

$$\text{Present worth second stage O\&M cost} = \frac{\text{Capitalized 2nd stage O\&M cost}}{(1+r)^n} \quad \text{(Eq. 4)}$$

$$\text{Present worth second stage pump set} = \frac{\text{Capitalized 2nd stage pump set}}{(1+r)^n} \quad \text{(Eq. 5)}$$

$$\text{PW of second stage pump set} = \frac{\text{Cost of 2nd stage pump sets}}{(1+r)^n}$$

11. Determine the total investment cost for the selected pipe size as given below:

Pipe size

a) cost of pipe length
b) 10% cost of pipe for the cost of specials
c) cost of pump sets for the first stage
d) capitalised cost of energy for the first stage

e) capitalised cost of o/m cost for the first stage
f) PW of capitalised energy cost for the second stage
g) PW of capitalised o/m cost for the second stage
h) PW of cost of pump sets for the second stage

A total of (a) to (h) is the total investment cost for the pipe size.

12. Repeat steps (4) to (11) by selecting the next available size. Compare with the total cost of the previous alternative, If the total cost is lesser than that of the previous size the process will be continued. Else the previously selected diameter is considered an economical diameter.

13. Prepare comparative statement for total investment cost for each selected size of pipes and select pipe size of least cost investment which is the most economic size of pumping main.

Excel worksheets can be programmed to simplify the calculation.

Economical diameter of rising main - Design steps

INPUT DATA

1) Water requirement :	Year	Discharge	
A. Initial	2012	5.00	mld
B. Intermediate	2027	7.50	mld
C. Ultimate	2042	10.00	mld
2) Pumping main	LENGTH	7000.00	M
3) Static head for pump	ST.HEAD	50.00	M
4) Design period	YEAR	30	yr.
5) Combined eff. of pump set	EFF. %	70.00	%
6) Cost of Pumping Set	Rs./KW	25000.00	Rs
7) MAAR=Interest Rate - Inflation rate	INTEREST	6.50	%
8) Life of elec. motors	P.Yrs	15	yr.
9) Energy charges /KWH Unit	Rs/KWH	5.00	Rupee
10) Pumping hours for discharge at the end of 15 Years	PUMPING-HOURS	22	hrs
11) Minor head loss as %age of frictional head loss		10.00	%
12) Standby pump capacity	percentage	50.00	%

13) PIPE DATA

Internal diameter in mm	Material	Class	HWC	Rate per m	OMR Rs/m
MM.	IAL			Rs/M	Rs/M
300.00	DI	K-9	140	3664.19	0.00
350.00	DI	K-9	140	4513.91	0.00
400.00	DI	K-9	140	5418.21	0.00
450.00	DI	K-9	140	6543.68	0.00
500.00	DI	K-9	140	7611.06	0.00

		1st 15 years	2nd 15 years
1)	Discharge at installation	5 MLD	7.5 MLD

2)	Discharge at the end 15 years	7.5 MLD	10.0 MLD
3)	Average discharge	5+7.5/2	7.5+10.0/2
4)	Average pumping hours during the period	(22/7.5) × 6.2522/10 × 8.75 = 18.33	= 19.25

5) KW required at combined efficiency

of pump set = $\frac{QH}{102\eta}$

Where:

Q = discharge at the end of 15 years in lps

H = Total head in m for discharge at the end of 15 years

η = combined efficiency of pumping set

$$KW1 = \frac{7.5x10^6 xH1}{22x3600x102x0.7} = 1.33H1 \qquad KW2 = \frac{10x10^6 xH2}{22x3600x102x0.7} = 1.77H2$$

6) Average annual charges for electrical energy

= KW × Average pumping Hours × Average days per year × Rate of energy charges per unit

= KW_1 × 18.33 × 365.25 × 5 = KW_2 × 19.25 × 365.25 × 5

= 33475.16 KW_1 = 35515.31 KW_2

7) Capitalisation factor for annual energy charges and OMR for converting them to the start of their periods

$$Cf = CR \left\{ \frac{1-(1+r)^{-n}}{r} \right\}$$

Where:

C_f = Capitalisation factor

C_R = Annual cost of energy charges + OMR

r = MARR, i.e., minimum attractive rate of return = 6.5%

n = period in year = 15

$$Cf = CR \left\{ \frac{1-(1+0.065)^{-15}}{0.065} \right\} = 9.4$$

8) Capitalisation factor for second stage pump cost (C_P)

$$C_P = \frac{1}{(1+r)^n} = \frac{1}{(1+0.065)^{15}} = 0.39$$

9) Capitalisation factor for the capitalised energy cost of the second stage to convert at the beginning of the first stage (Ce)

$$Ce = \frac{1}{(1+r)^n} = \frac{1}{(1+0.065)^{15}} = 0.39$$

TABLE 1 -VELOCITY AND LOSS OF HEAD FOR DIFFERENT PIPE SIZES

Sr.No.	Internal Pipe diameter (mm)	Frictional Head loss per 1000 m		Velocity (m/s)		Total head in 'm' for 7000 m pipe length including 50 m of static head					
		1st stage flow 7.5 MLD	2nd stage flow MLD	1st stage flow	2nd stage flow	Frictional loss 1st stage flow	Minor head loss for 1st stage flow	Total head for pumping H1 for 1st stage flow	Frictional loss 2nd stage flow	Minor head loss for 2nd stage flow	Total head for pumping H1 for 2nd stage flow
1	2	3	4	5	6	7	8	9	10	11	12
1	300.00	5.06	8.63	1.34	1.79	35.45	3.55	89.00	60.40	6.04	116.44
2	350.00	2.39	4.07	0.98	1.31	16.73	1.67	68.41	28.51	2.85	81.36
3	400.00	1.25	2.13	0.75	1.00	8.73	0.87	59.61	14.88	1.49	66.37
4	450.00	0.70	1.20	0.60	0.79	4.92	0.49	55.41	8.38	0.84	59.22
5	500.00	0.42	0.72	0.48	0.64	2.95	0.29	53.24	5.02	0.50	55.52

>>> * Minor head loss = 10% of frictional loss

Table (3)

Column 2 = Cost of pump sets

Column 3 = Annual cost of energy charges and OMR

= (33475.16 KW_1 + OMR) = CR_1

Column 4 = Capitalised cost of energy charges and OMR for the first stage

= $Cf \times CR_1 = 9.4 \times$ Column (3) of Table (3)

Column 5 = Total capitalised cost for first stage

= Column (11) of Table (2) + Column (2) of Table (3) + Column (4) of Table (3)

Column 6 = Cost of pump set for second stage

Column 7 = Annual energy and OMR cost

= (35155.31 KW_2 +OMR) = CR_2

Column 8 = Capitalised cost of energy and OMR cost for second stage

= $C_f \times CR_2$

= 9.4 × (Column (7) of Table (3))

Column 9 = Initial capital investment for annual energy charges, OMR changes and cost of replacement of pump set for the second stage

= (Column (6) of Table (3) + Column (8) of Table (3)) × Capitalisation factor for second stage pump cost (C_P) as calculated at step No 8

Column 10 = Grand Total of capitalised cost for 30 years

= Column (5) of Table (3) + Column (9) of Table (3)

TABLE 2 - KILOWATTS & COST OF PUMP SETS REQUIRED FOR DIFFERENT PIPE SIZES AND PIPE COST

			1st stage flow of 7.5 million liters/day			2nd stage flow of 10 million liters/day				
SI.No.	**Internal Pipe diameter (mm)**	Class of pipe	H1 Total head (m)	Total working KW required	Pump Cost including cost of stand by sets as mentioned at input data at Sr No 12 @ Rs 25000 per KW (Rs in thousand)	H2 Total head (m)	Total working KW required	Pump Cost including cost of stand by sets as mentioned at input data at Sr No 12 @ Rs 25000 per KW (Rs in thousand)	Cost of pipe per unit length (Rs)	Cost of 7000 meter pipe line (Rs. In thousand)
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	300.00	K-9	89.00	118.00	4425.00	116.00	205.00	7688.00	3664.19	25649.33
2	350.00	K-9	68.00	90.00	3375.00	81.00	143.00	5363.00	4513.91	31597.37
3	400.00	K-9	60.00	80.00	3000.00	66.00	117.00	4388.00	5418.21	37927.47
4	450.00	K-9	55.00	73.00	2738.00	59.00	104.00	3900.00	6543.68	45805.76
5	500.00	K-9	53.00	70.00	2625.00	56.00	99.00	3713.00	7611.06	53277.42

TABLE 3 - COMPARATIVE STATEMENT OF OVERALL COST OF PUMPING MAIN FOR DIFFERENT PIPE SIZES

Sl. No	Capital cost of pump sets including cost of standby sets @ Rs 25000 per KW (Rs in thousand)	Annual Energy & OMR cost for first stage(Rs. thousand)	Capitalised cost of Annual Energy & OMR for first stage (Rs. in thousand)	Total Capitalised cost of first stage (Rs. in thousand)	Capital cost of pump sets including standby sets @ Rs 25000 per KW (Rs in thousand) for second stage	Annual Energy & OMR cost for second stage (Rs. thousand)	Capitalised cost of Annual Energy & OMR for second stage (Rs. in thousand)	Initial capital investment for pump sets and annual Energy and OMR cost for second stage (Rs in thousand)	Grand Total of Capitalised cost for 30 yrs (Rs. in thousand)	**Internal Pipe diameter (mm)**
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	4425	3950	37141	67215	7688	7207	67765	29338	96553	300.00
2	3375	3013	28330	63302	5363	5027	47267	20464	83766	350.00
3	3000	2678	25180	66107	4388	4113	38673	16743	82850	400.00
4	2738	2444	22980	71524	3900	3656	34376	14883	86407	450.00
5	2625	2343	22030	77932	3713	3480	32721	14167	92099	500.00

Remarks : From this table, it is seen that most economical diameter of main is 400 mm having least capitalised cost.

Note : 1) Total KW and capital cost of pumping machinery shall be calculated including cost of standby set as mentioned at Sr No 12 of input data.

2) Annual energy charges shall be calculated for working KW .

Annexure 6.2: Grouping High-Level and Low-Level Tanks

As shown above, grouping high-level and low-level service reservoirs (ESRs) in the city (Aurangabad) should be done using GIS tools like IDW surface. In the GIS software, using GIS-based contours, the IDW surface is created.

Water should be supplied to high-level and low-level service reservoir (ESRs) areas by MBRs located at different elevations. This is necessary to optimise the pumping cost.

One city in Maharashtra is getting water supply from the source of a major irrigation dam. Water from the dam is pumped to WTP and after treatment, it is again pumped to the MBR at one hillock. Before 2003, the old transmission network (which is shown within circumscribing the area with a dotted red line in Figure 1, was supplying water by gravity. Water was supplied from the existing MBR at LSL of 632 m. This old transmission main touches the lowest level of the city, as may be seen in the darkest green area at RL 538 m.

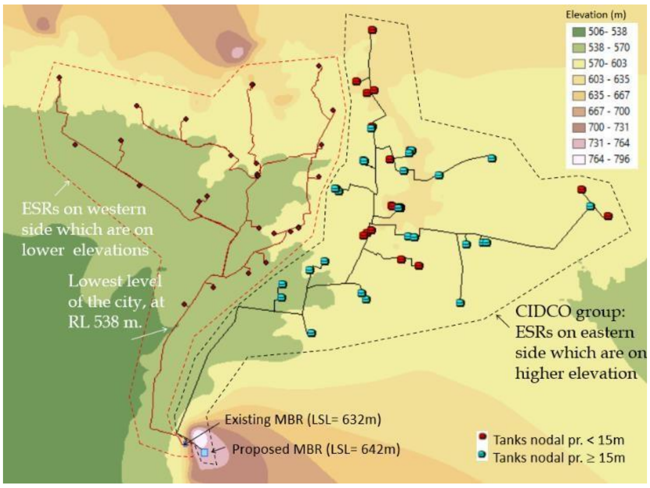


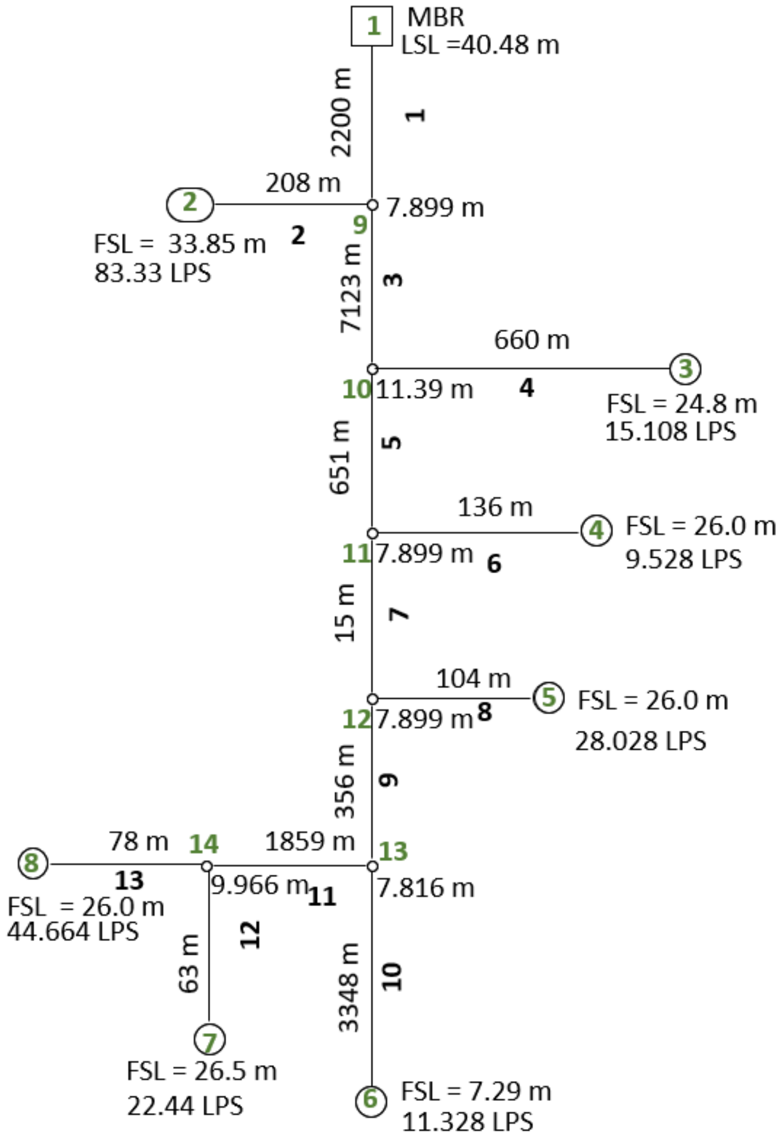
Figure 1: Old transmission network in red and new transmission network (2003) in grey colour

Part of the water was being pumped from the lowest point to the high-level ESRs. Eventually, ESRs located on the eastern side of the city, having higher elevations were the sufferers till the year 2003. The practice of bringing the total quantum of water to the city centre and then to distribute it further without consideration to topography and optimisation of energy cost, should not be adopted.

Solution: New scheme was commissioned in 2003 to mitigate the growing water demand and to redress the problem of ESRs located in higher elevation areas shown by circumscribing with black dotted line as shown in Figure 1, by laying separate transmission main along the higher contour. This arrangement has solved the problem, and in the process, energy cost is also minimised.

Annexure 6.3: Optimisation of Gravity Transmission Main

One complete gravity water supply scheme of a city is considered for the sake of illustration.



The source of water is located at a higher level and after treatment water is stored in MBR from where it can be supplied by gravity to all the city reservoirs. The LSL of MBR is 40.48 m. MBR is supplying water to seven reservoirs labelled as 2 to 8. Other intermediate junctions are labelled from 9 to 14. The FSLs of reservoirs and ground elevation at junction nodes are given in Table 1. Nodal demands are also given in 1. These demands are increased by a factor of 1.058 to include the minor losses. Pipes are labelled 1 to 13 and their connectivity and lengths are given in Table 2. Available pipe sizes in mm and their unit costs in Rs/m length are given in Table 3.

The network is designed using JalTantra, considering the minimum residual head requirement of 3 m above the FSL of each reservoir.

Table 1: Node Data		
Node ID	Elevation, m	Demand, L/s
2	33.85	83.33
3	24.8	15.108
4	26	9.528
5	26	28.028
6	27.29	11.328
7	26.5	22.44
8	26	44.664
9	7.899	
10	11.39	
11	7.899	
12	7.899	
13	7.816	
14	9.966	

Table 2: Pipe Data			
Pipe ID	Start Node	End Node	Length, m
1	1	9	2,200
2	9	2	208
3	9	10	7,123
4	10	3	660
5	10	11	651
6	11	4	136
7	11	12	15
8	12	5	104
9	12	13	356
10	13	6	3,348
11	13	14	1,859
12	14	7	63
13	14	8	78

Table 3: Commercial Pipe Data		
Diameter	Roughness	Cost (Rs/m)
80	140	750
100	140	952
150	140	1,383
200	140	1,870
250	140	2,485
300	140	3,106
350	140	3,919
400	140	4,558
450	140	5,560
500	140	6,405
600	140	8,175
700	140	10,370

Table 4: General Data	
Network Name	MBR
Organisation Name	CPHEEO
Minimum Node Pressure	3
Default Pipe Roughness 'C'	140
Minimum Head loss per KM	0
Maximum Head loss per KM	500
Maximum Water Speed	2.5
Number of Supply Hours	24
Source Node ID	1
Source Elevation	40.48
Source Head	40.48

Table 5: NODE OUTPUT					
Node ID	Demand (L/s)	Elevation (m)	Head (m)	Pressure (m)	Min. Pressure (m)
2	69.44	33.85	36.85	3	3
3	12.59	24.8	27.8	3	3
4	7.94	26	29	3	3
5	23.36	26	29	3	3
6	9.44	27.29	30.29	3	3
7	18.7	26.5	29.5	3	3
8	37.22	26	29	3	3
9	0	7.9	39.23	31.33	3
10	0	11.39	32.71	21.32	3
11	0	7.9	32.23	24.33	3
12	0	7.9	32.22	24.32	3
13	0	7.82	32	24.18	3
14	0	9.97	29.54	19.57	3

Table 6: PIPE OUTPUT										
Pipe ID	Start Node	End Node	Length (m)	Flow (LPS)	Speed (m/s)	Diameter (mm)	Roughness 'C'	Head loss (m)	Head loss (m/KM)	Cost (Rs)
			m	L/s	m/s	mm		m	m/Km	Rs.
1	1	9	13	178.69	0.91	500	140	0.018	1.364	85,570
	1	9	2,187	178.69	0.632	600	140	1.227	0.561	1,78,75,783
2	9	2	69	69.44	2.21	200	140	1.419	20.539	1,29,172
	9	2	139	69.44	1.415	250	140	0.963	6.928	3,45,227
3	9	10	7,123	109.25	0.687	450	140	6.526	0.916	3,96,03,880
4	10	3	118	12.59	1.603	100	140	2.992	25.421	1,12,053
	10	3	542	12.59	0.712	150	140	1.914	3.529	7,49,997
5	10	11	651	96.66	0.608	450	140	0.475	0.73	36,19,560
6	11	4	83	7.94	1.58	80	140	2.653	32.089	62,012
	11	4	53	7.94	1.011	100	140	0.577	10.825	50,759
7	11	12	15	88.72	0.558	450	140	0.009	0.623	83,400
8	12	5	30	23.36	2.974	100	140	2.401	79.865	28,625
	12	5	74	23.36	1.322	150	140	0.82	11.086	1,02,248
9	12	13	356	65.36	0.52	400	140	0.224	0.628	16,22,648
10	13	6	3,348	9.44	0.3	200	140	1.707	0.51	62,60,760
11	13	14	777	55.92	0.791	300	140	1.484	1.909	24,14,860
	13	14	1,082	55.92	0.581	350	140	0.975	0.901	42,38,468

12	14	7	63	18.7	0.381	250	140	0.038	0.61	1,56,555
13	14	8	2	37.22	2.106	150	140	0.045	26.27	2,352
	14	8	76	37.22	1.185	200	140	0.494	6.471	1,42,680

Table 7: COST RESULTS			
Diameter	Length	Cost	Cumulative Cost
80	83	62,012	62,012
100	201	1,91,437	2,53,448
150	618	8,54,596	11,08,044
200	3,493	65,32,612	76,40,656
250	202	5,01,782	81,42,438
300	777	24,14,860	1,05,57,298
350	1,082	42,38,468	1,47,95,766
400	356	16,22,648	1,64,18,414
450	7,789	4,33,06,840	5,97,25,254
500	13	85,570	5,98,10,824
600	2,187	1,78,75,783	7,76,86,607
Total	16,801	7,76,86,607	

Remarks

1. Some of the links as shown in pipe data (Table 6) are of two sizes. For example, pipe 1 of 2,200 m in length is split into two sizes of 600 mm and 500 mm having a length of 2,187 m and 13 m, respectively.
2. Small lengths of split sizes like 13 m in link 1 of 500 mm can be changed to a higher size to avoid a reducer. This will, however, result in the excess head at one or more reservoirs which will then be required to be dissipated by providing head-dissipating devices.
3. Incidentally, dropping 500 mm size, will not require any procurement of that size as can be seen from Table 6.3.7.

Annexure 6.4: Direct Pumping Main System (Nadia)

A Case Study: RWSS In Nadia District of West Bengal

A Pumped Water Transmission System from WTP to nine village reservoirs is a grouped RRWSS is shown below. The system cost is optimised by considering both costs of pipes and the cost of energy. (Alternative methodology is given in the Case Study at Annexure 16.2 where pumping in different directions from single sump by choosing appropriate location is narrated along with its benefits.)

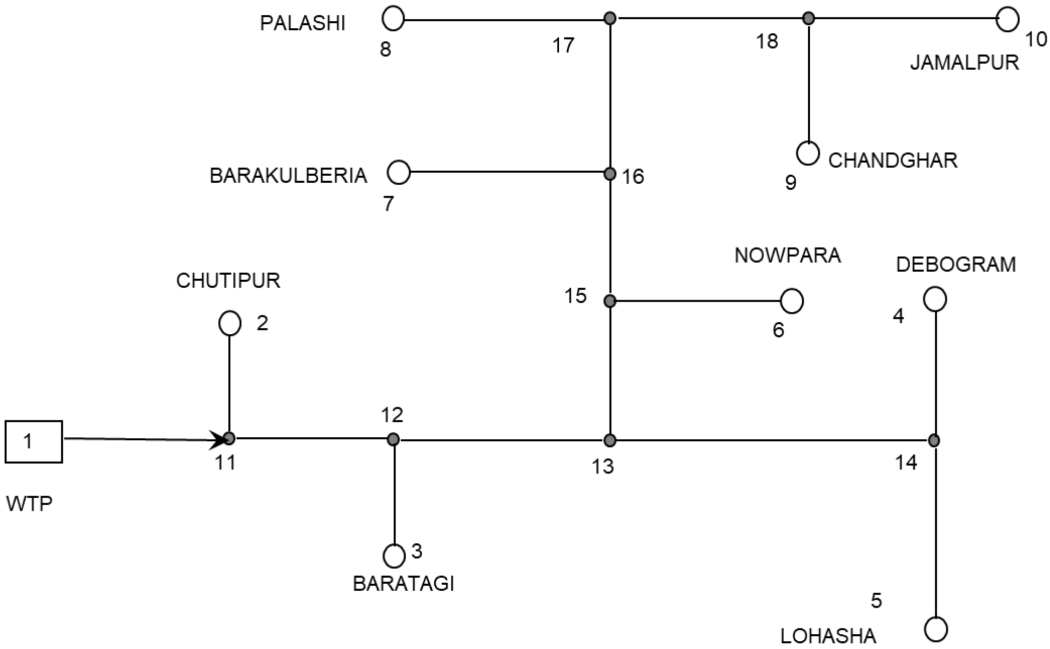


Figure 1. Network Line diagram showing Layout connecting various ESRs connected to Sump and Pump at WTP.

Design Data

1) Design period = 30 years
2) Hours of pumping in a day = 20
3) Cost of pump = Rs. 10000 per kW
4) Cost of power = Rs. 2.5 per kWh
5) Pump efficiency = 70 %
6) Life of pump = 15 years
7) Interest rate = MARR = 8 %
8) Pipe Material = DI K-9
9) Hazen-Williams Coefficient for DI-K9 pipe = 140
10) Standby pump capacities = 50%
11) Minor head loss = 10% of Frictional head loss

To account for minor head losses as 10% of frictional head loss, nodal demands are increased by an appropriate factor (=1.0528).

Pipe Cost:

Pipe material, available pipe sizes, and cost per metre length of pipe are given in Table 1.

Table 1: Available pipe sizes and their unit cost

No.	Pipe Material	Diameter in mm	Cost Rs/m length
1	DI-K-9	80	270
2	DI-K-9	100	520
3	DI-K-9	150	790
4	DI-K-9	200	1,090
5	DI-K-9	250	1,430
6	DI-K-9	300	1,810
7	DI-K-9	350	2,240
8	DI-K-9	400	2,730
9	DI-K-9	450	3,267
10	DI-K-9	500	3,810
11	DI-K-9	600	5,016
12	DI-K-9	700	6,561

Network Description and Pipe/Node Data:

LSL and FSL of village reservoirs are fixed considering the distribution network of the concerned village. GL at various nodes and FSLs at village reservoirs labelled 2 to 10 are shown in Table 2. Nodes 11 to 18 are junctions with no water demand. The FSLs are the actual elevations of the supply points to the reservoirs. For example, the FSL at Chutipur village reservoir is 42.65 m. The residual pressure requirement at each village reservoir is 3 m.

Design nodal demands are given in column 4 at all the reservoirs. These demands are worked out by considering 20 hrs of daily pumping. Further, to include minor losses, these values are multiplied by 1.0528. These values are 0 at junction nodes having no demand. Pipe numbers and pipe lengths are given in columns 5 and 6 of Table 2.

It is proposed to feed all the reservoirs simultaneously with a pump located at one place in pipe 1 as the deviation in ground levels as well as in FSLs are not much at different reservoirs.

Table 2: Node and pipe data for the illustrative pumped transmission network

Node number	LWL/GL	FSL	Peak demand x 1.0528	Pipe number	Length
	m	m	L/s		m
(1)	(2)	(3)	(4)	(5)	(6)
Sump at WTP	10.139			1	6972
Pump	13.139			2	1500
2	13.650	42.65	40.775	3	3,385.55
3	13.025	42.025	42.407	4	1300
4	14.659	43.659	40.080	5	2,868.17
5	12.500	41.50	35.048	6	4508
6	13.800	42.80	33.311	7	2,127.86
7	13.130	42.13	38.153	8	3100
8	12.110	41.11	42.428	9	1,545.28
9	14.360	43.36	55.819	10	1,900.34
10	12.550	41.55	28.699	11	7,293.07
11	12.960			12	200
12	14.120			13	3620
13	14.290			14	50
14	14.130			15	6,404.32
15	15.280			16	460.44
16	14.66			17	7,752.98
17	13.72				
18	15.40				

Design Steps

1. Fixing a range for HGL at pumped source node using critical path method.

   A critical path is a path having the least available frictional slope (=Available head difference/length of the path). Various paths from the source to different reservoir nodes are listed in column 1 of Table 3. As the HGL at the pump source node is not known, a suitable value is assumed just to identify a critical node. Herein, this value is taken as 100 as shown in column 2. The minimum required elevation at the end of the path is mentioned in column 3. The total path length from source to demand nodes is given in column 4. The available head difference on each path is obtained as shown in column 5 by considering the residual pressure head requirement of 3 m at each node (column 5 = column 2 - column 3). Finally, available hydraulic slopes are obtained as shown in column 6 (=column 5/column 4). It can be observed that path1-10 has a minimum available slope of 1.392 m/km, hence critical.

Thus, node 10 is most critical. Therefore, the range of HGL is decided based on this node FSL, required residual pressure, and average head loss on this between 1.5 m/km to 2 m/km.

The minimum selected HGL at pump source node = 41.55+3+1.5×39.34 = 103.5; the maximum selected value = 41.55+3+2×39.34 = 123.2 m. So, initially, HGL values in a broader range of 100-125 m, at 2 m intervals, are considered. When it seems that changes in the optimal cost are very less and likely to increase on increasing the HGL, the interval is reduced to 1 m.

Table 3: Identification of critical path

Path	Assumed HGL at the Source, m	Minimum required elevation at the end of the path (m)	Total Path Length, km	Available head difference considering 3 m residual head, m	Available hydraulic slope m/km
(1)	(2)	(3)	(4)	(5)	(6)
1-2	100	42.65	8.472	54.35	6.415
1-3	100	42.03	11.658	54.97	4.715
1-4	100	43.66	19.862	53.34	2.686
1-5	100	41.5	20.834	55.5	2.664
1-6	100	42.8	16.671	54.2	3.251
1-7	100	42.13	22.264	54.87	2.465
1-8	100	41.11	25.734	55.89	2.172
1-9	100	43.36	32.549	53.64	1.648
1-10	100	41.55	39.841	55.45	1.392

2. Considering the maximum FSL of the service reservoir and head loss in the pipe, the pumping head is decided between 100 to 124m, at an interval of 2 m, to generate various alternatives. For each alternative, JalTantra, which uses the LP technique for branched network design, is used to obtain the cost of the network. Further, the same HGL pump cost and energy cost for two different periods are obtained in Excel sheet as given below. Results in the table are shown for HGL from 96 m onwards as some sensitivity analysis is also carried out.
3. The format of the Excel sheet is shown in Table 4. Values in blue colours are input values to compute pump and energy costs.

Table 4: Calculation of optimal HGL at the pump node to minimise the total cost

1	LSL of sump in metres		10.139
2	Hours of pumping		20

3	Pump efficiency in percentage		70
4	Pump life, n in years		15
5	Pump cost in 1000Rs/KW		10
6	Energy cost, Rs/kWh		2.5
7	Rate of interest, i×100		8
8	FSL of critical reservoir		41.55
9	Pump standby units in percentage		50
	Single payment PW Factor =	$1/(1+i)^n$	0.3152
	Uniform Series PW Factor =	$((1+i)^n-1)/i\times(1+i)^n$	8.5595

Rows (1) to (9) show the input values. As the pump life is 15 years and the maximum attractive rate of return is the same as the interest rate of 8 per cent, the single payment PW factor for converting pump replacement cost to its PW and annual energy charges to their PW are obtained. The single payment PW factor is 0.3152 and the uniform series PW factor is 8.5595.

Table 5: Demand rate at different stages, average rate of demand and discharge and head loss ratio

Design Demand	Initial Demand	Inter Demand	Avg Q I Period	Avg Q II Period	Avg Hours I Period	Avg. Hours II Period
LPS	**LPS**	**LPS**	**LPS**	**LPS**		
338.83	219.90	279.53	249.718	309.182	17.867	18.25
1	0.6490	0.825	0.7370	0.9125	**Discharge Ratio (DR)**	
1.000	0.449	0.700	0.568	0.844	**Head loss Ratio**	

From the ultimate stage design demand (no multiplication by 1.0528), the average of 1st and 2nd periods are obtained. Further, discharge ratios with design demands, i.e., Q/Q^{des}, are obtained, and further using them head loss ratios are obtained. Head loss is $h = RQ^{1.852}$, assuming all conditions same, the head loss ratio will be (discharge ratio)$^{1.852}$. These values will be useful later in the computation of pump capacity and energy costs for different periods.

4. As discussed earlier, source HGL from 96 to 124 m is considered as shown in column (1) of Table 4.2. Next, the pumping head is calculated by subtracting the LSL from HGL at the source. HGL requirement at the source node is for two purposes, (1) to overcome the elevation difference; and (2) to overcome the frictional loss. Total static lift on the critical path is obtained and head loss on the critical path is obtained. This is necessary to be separated for direct computation of pump power and average energy power requirement. Say, when the demand is less in the first period, the pump will work at a lower head as the head loss will also be less. However, the static lift will always remain constant. Therefore, the reduced head loss can be computed by applying head loss ratios to head loss terms only.

With a source of HGL of 108 m (Col. 1) and LSL of 10.139 m (Col. 2), the pumping head is 97.681 m (Col. 3) [Col. 3=Col.1 - Col.2]. As the FSL of the critical node is 41.55 m and LSL is

10.139 m, the static lift is 31.411 m (Col. 4), and head loss on the critical path is 66.45 m, Col.5. [Col.3-Col.4=Col.5].

5. The cost of pumps and energy charges over the 30-year design period is then calculated and their PW is obtained as shown in next Tables 4.3 and 4.4, respectively.

Table 6: Computation of pumping head and head loss on a critical path

Source HGL m	Lowest Suction Level m	Pumping Head m	Static Lift m	Head loss on critical path
(1)	(2)	(3)	(4)	(5)
96	10.139	85.861	31.411	54.45
98	10.139	87.861	31.411	56.45
100	10.139	89.861	31.411	58.45
102	10.139	91.861	31.411	60.45
104	10.139	93.861	31.411	62.45
106	10.139	95.861	31.411	64.45
108	10.139	97.861	31.411	66.45
110	10.139	99.861	31.411	68.45
112	10.139	101.861	31.411	70.45
114	10.139	103.861	31.411	72.45
116	10.139	105.861	31.411	74.45
118	10.139	107.861	31.411	76.45
120	10.139	109.861	31.411	78.45
121	10.139	110.861	31.411	79.45
122	10.139	111.861	31.411	80.45
123	10.139	112.861	31.411	81.45
124	10.139	113.861	31.411	82.45

Table 7: Computation of PW of total pump cost

Pump Power I Period KW	Pump Cost I Period 1000 Rs	Pump Power II Period KW	Pump Cost II Period 1000 Rs	PW of Pump Cost II Period 1000 Rs	PW of Total Pump Cost 1000 Rs
(1)	(2)	(3)	(4)	(5)	(6)
272.43	4086.40	407.71	6115.61	1927.90	6014.30
277.91	4168.70	417.20	6258.07	1972.80	6141.51
283.40	4251.00	426.70	6400.52	2017.71	6268.71
288.89	4333.30	436.20	6542.97	2062.62	6395.92
294.37	4415.60	445.70	6685.43	2107.53	6523.13
299.86	4497.90	455.19	6827.88	2152.43	6650.34
305.35	4580.20	464.69	6970.34	2197.34	6777.54
310.83	4662.50	474.19	7112.79	2242.25	6904.75
316.32	4744.80	483.68	7255.24	2287.16	7031.96
321.81	4827.10	493.18	7397.70	2332.06	7159.17
327.29	4909.40	502.68	7540.15	2376.97	7286.37
332.78	4991.70	512.17	7682.61	2421.88	7413.58
338.27	5074.00	521.67	7825.06	2466.78	7540.79
341.01	5115.15	526.42	7896.29	2489.24	7604.39
343.75	5156.30	531.17	7967.51	2511.69	7668.00
346.50	5197.45	535.92	8038.74	2534.15	7731.60
349.24	5238.60	540.66	8109.97	2556.60	7795.20

Pumps are designed for peak discharge at the end of periods. The pump power in KW is obtained by using the formula as $9.81 \times Q \times (H_s + h_L)/\eta$. In the first row, the intermediate stage demand is 279.535 L/s which is converted to cubic m/s for use. The head loss is reduced by multiplying the head loss (66.45 m) by the head loss ratio (0.7) as the pumps are designed to operate at lower heads in the first 15 years. The pump power is 305.35 KW (Col.1). Its cost is worked out by multiplying it per KW pump cost and 50% standby capacity is considered. Results are depicted in terms of 1,000 Rs (Col. 2). Similarly, the capacity requirement for the second stage is worked out, and pump cost is calculated assuming 50% standby (Cols. 3 and 4). The PW of the second period is obtained using the single payment PW factor (Col. 5). PW of total pump cost is obtained in addition to costs of the first and second periods (Col. 6).

Similarly, the PW of energy costs for the two different 15 years periods is obtained. Energy costs are calculated based on average flows in the two periods. The number of hours of operations is obtained for average flows. In the first column, energy consumption per kWh is shown. Average annual energy costs are calculated considering the average hours of pumping in a day, the number of days in a year, and unit energy cost (Col. 2). PW of annual energy cost for the first period is shown in Col. 3. Similarly, PW of energy cost is obtained in Cols. 4 to 6. The PW of energy cost requires both conversions from annual to one-time cost at the beginning of the second period and then the same at the initial. PW of the total cost is given in Col. (7).

Table 8: Computation of PW of total energy cost

Energy Required I Period kWh	Avg. Annual Energy cost I period 1000 Rs	PW of Energy cost I period 1000 Rs	Energy Required II Period kWh	Avg. Annual Energy cost II period 1000 Rs	PW of Energy cost II period 1000 Rs	PW of Total Energy Cost 1000 Rs
(1)	(2)	(3)	(4)	(5)	(6)	(7)
272.43	4444.51	38042.66	407.71	6794.26	18332.97	56375.63
277.91	4534.02	38808.84	417.20	6952.52	18760.01	57568.85
283.40	4623.53	39575.02	426.70	7110.78	19187.05	58762.07
288.89	4713.04	40341.20	436.20	7269.04	19614.09	59955.28
294.37	4802.56	41107.38	445.70	7427.30	20041.12	61148.50
299.86	4892.07	41873.56	455.19	7585.56	20468.16	62341.72
305.35	4981.58	42639.74	464.69	7743.83	20895.20	63534.94
310.83	5071.09	43405.92	474.19	7902.09	21322.24	64728.16
316.32	5160.61	44172.10	483.68	8060.35	21749.28	65921.37
321.81	5250.12	44938.28	493.18	8218.61	22176.32	67114.59
327.29	5339.63	45704.45	502.68	8376.87	22603.35	68307.81
332.78	5429.14	46470.63	512.17	8535.13	23030.39	69501.03
338.27	5518.66	47236.81	521.67	8693.40	23457.43	70694.25
341.01	5563.41	47619.90	526.42	8772.53	23670.95	71290.85
343.75	5608.17	48002.99	531.17	8851.66	23884.47	71887.46
346.50	5652.92	48386.08	535.92	8930.79	24097.99	72484.07
349.24	5697.68	48769.17	540.66	9009.92	24311.51	73080.68

Next, the PW of pipe cost (Col. 2), pump cost (Col. 3), and energy cost (Col. 4) are added to obtain the PW of the total cost (Col. 5) for each alternative HGL at the pump source (Col. 1) in Table 4.5. Pipe network cost is obtained using JalTantra software for each alternative. It can be observed that pipe cost decreases with an increase in HGL at the source, and pump cost and energy cost increase with an increase in HGL. The sum of the total cost is observed to be continuously decreasing from 96 to 108 m of HGL at the pump source. It can be observed increasing from 110 m onwards. Thus, an HGL of 108 m is considered optimal. For more accuracy, some values near 108 m can be tried, or a graph on an excel sheet can be generated to select the optimal HGL value.

The optimal solution is sensitive to various parameters used in the study. For example, if energy cost is considered as Rs. 3 per kWh instead of Rs. 2.5 per kWh, the optimal HGL will be observed at 100 m. The optimal HGL was obtained at 100 m, for energy charges of Rs. 2 per kWh. Thus, as the energy cost increase, the optimal pumping head reduces which increases the pipe costs. Energy charges get over calculated when a single period of 30 years is considered, the optimal solution will be erroneous.

Table 9: PW of Total Cost for different alternative HGL at Pump Source

Source HGL m	Pipe Cost M Rs	PW of pump cost M Rs	PW of energy cost M Rs	PW of total cost in M Rs
96	161.626	6.014	56.376	224.016
98	159.849	6.142	57.569	223.560
100	158.106	6.269	58.762	223.137
102	156.665	6.396	59.955	223.016
104	155.250	6.523	61.149	222.921
106	153.840	6.650	62.342	222.832
108	152.467	6.778	63.535	222.780
110	151.203	6.905	64.728	222.836
112	149.946	7.032	65.921	222.899
114	148.689	7.159	67.115	222.963
116	147.457	7.286	68.308	223.052
118	146.243	7.414	69.501	223.158
120	145.105	7.541	70.694	223.340
121	144.536	7.604	71.291	223.431
122	143.969	7.668	71.887	223.525

123	143.447	7.732	72.484	223.662
124	142.924	7.795	73.081	223.800

The final design with an HGL of 108 m as obtained using JalTantra is shown in Table 6. Also, as JalTantra considers the single period of 30 years in directly determining the optimal pipe sizes and pumping head, a solution is obtained using direct application of JalTantra on a fixed layout by allowing pump location in pipe 1. The optimal pumping head is obtained as 82.04 m with optimal HGL at the pump source as 92.179 m. The corresponding network and pumping cost that includes both pump and energy costs is given in Table 6. It can be observed that both pipe cost and pumping cost are on the higher side. Even though pump replacement cost is not considered, the energy charges are considered on a higher side. Hence, direct application of JalTantra for city water supply pumping is not recommended. It can be used as discussed earlier for the design of the network by fixing HGL at the pump source and comparing various alternatives.

Table 10: Design output file from JalTantra for pipe sizes with HGL of 108 m at pump source

Network Name	NADIA
Organisation Name	Abbas for CPHEEO
Minimum Node Pressure	0
Default Pipe Roughness 'C'	140
Minimum Head loss per KM	0
Maximum Head loss per KM	10,000.00
Maximum Water Speed	4.5
Maximum Pipe Pressure	
Number of Supply Hours	24
Source Node ID	1
Source Node Name	MBR
Source Elevation	10.14
Source Head	108
Number of Nodes	18
ESR Enabled	FALSE
Pump Enabled	FALSE
Total Length of Network	54,988
Total Length of New Pipes	54,988
Total Pipe Cost	152,467,141

NODE RESULTS						
Node ID	Node Name	Demand	Elevation	Head	Pressure	Min. Pressure
2	Node2	40.775	45.65	47.26	1.61	0
3	Node3	42.407	45.03	45.03	0	0
4	Node4	40.08	46.66	46.66	0	0
5	Node5	35.048	44.5	44.5	0	0
6	Node6	33.311	45.8	45.8	0	0
7	Node7	38.153	45.13	57.44	12.31	0
8	Node8	42.428	44.11	53.65	9.54	0
9	Node9	55.819	46.36	46.36	0	0
10	Node10	28.699	44.55	44.55	0	0
11	Node11	0	12.96	93.92	80.96	0
12	Node12	0	14.12	88.46	74.34	0
13	Node13	0	14.29	79.85	65.56	0
14	Node14	0	14.13	62.45	48.32	0
15	Node15	0	15.28	77.29	62.01	0
16	Node16	0	14.66	62.94	48.28	0
17	Node17	0	13.72	55.33	41.61	0
18	Node18	0	15.4	48.85	33.45	0

PIPE RESULTS										
Pipe ID	Start Node	End Node	Length	Flow	Diameter	Roughness 'C'	Head loss	Head loss per KM	Cost	Speed
1	1	11	6,972	356.72	600	140	14.08	2.02	3,49,71,552	1.262
2	11	2	1,500	40.78	150	140	46.657	31.105	11,85,000	2.307
3	11	12	3,386	315.95	600	140	5.461	1.613	1,69,81,919	1.117
4	12	3	1,298	42.41	150	140	43.411	33.45	10,25,254	2.4
4	12	3	2	42.41	200	140	0.018	8.24	2,409	1.35
5	12	13	2,868	273.54	500	140	8.608	3.001	1,09,27,728	1.393
6	13	14	535	75.13	250	140	4.292	8.016	7,65,694	1.53
6	13	14	3,973	75.13	300	140	13.104	3.299	71,90,314	1.063
7	14	4	2,128	40.08	200	140	15.794	7.422	23,19,367	1.276
8	14	5	0	35.05	150	140	0.009	23.501	288	1.983
8	14	5	3,100	35.05	200	140	17.945	5.79	33,78,602	1.116
9	13	15	1,545	198.41	500	140	2.559	1.656	58,87,517	1.01
10	15	6	1,332	33.31	150	140	28.499	21.39	10,52,544	1.885
10	15	6	568	33.31	200	140	2.993	5.269	6,19,126	1.06
11	15	16	7,293	165.1	450	140	14.354	1.968	2,38,26,460	1.038
12	16	7	200	38.15	150	140	5.5	27.502	1,58,000	2.159
13	16	17	3,449	126.95	400	140	7.405	2.147	94,16,000	1.01

13	16	17	171	126.95	450	140	0.207	1.21	5,58,382	0.798
14	17	8	50	42.43	150	140	1.674	33.48	39,500	2.401
15	17	18	6,404	84.52	400	140	6.473	1.011	1,74,83,794	0.673
16	18	9	40	55.82	200	140	0.55	13.708	43,704	1.777
16	18	9	420	55.82	250	140	1.944	4.624	6,01,092	1.137

COST RESULTS OF NEW PIPES			
Diameter	Length	Cost	Cumulative Cost
150	4,380	34,60,586	34,60,586
200	5,838	63,63,209	98,23,796
250	956	13,66,787	1,11,90,582
300	11,726	2,12,23,208	3,24,13,791
400	9,853	2,68,99,794	5,93,13,584
450	7,464	2,43,84,841	8,36,98,426
500	4,413	1,68,15,245	10,05,13,670
600	10,358	5,19,53,471	15,24,67,141
Total	54,988	15,24,67,141	

Table 6. Optimal solution by JalTantra considering a single period of 30 years

Total Pipe Cost	16,57,74,703
Total Pump Cost	11,32,73,668
Total Cost	27,90,48,371

The proposed method has the advantage that it considers pump cost, energy cost, and pipe cost in deciding the diameters of different pipes and the selection of pumps. The following can be observed in Table 6.

1. It can be noticed from the pipe details that pipes 4, 6, 8, 10, 13, and 16 have split sizes. For pipes 4 and 8, lengths of 150 mm size pipes as given by software are 2m and less than 0.5 m (rounding off to nearest whole number 0). From a practical point of view, a split pipe of less than 5 m length can be provided of higher size as this will save on the cost of the reducer. However, this may cause little excess head.
2. The minimum available pipe size was 80 mm; however, the software has not chosen it even when extra residual pressure was available at a few of the nodes. This is due to velocity restrictions in pipes to 4.35 m/s.
3. The velocity ranges from 0.678 to 2.4 m/s in different pipes. It is necessary that the design is checked for water hammer pressure and if necessary, suitable devices are provided to control the water hammer pressure.
4. No excess residual head above 3 m is available at reservoir nodes 3, 4, 5, 6, 9, and 10 as can be seen from node details. However, the excess residual head is available at reservoirs 2, 7, and 8.
5. The excess residual head needs to be dissipated to match the field performance with the designed one. However, in this case, replacement of part or the full length of any

pipe with a smaller size cannot be done, as the velocity in the pipeline will increase beyond the permissible limit. As discussed earlier, the extra head can be dissipated by providing the following devices alone or in combination - orifice plates, valves of the same or smaller size, and its throttling. Such a head dissipation system can be designed as explained in the Manual on operation and maintenance of water distribution system. The Chapter on “Head Dependent Analysis” from the book “Analysis of Water Distribution Networks” by P. R. Bhave can also be referred for the design of the head dissipation system. It is further emphasised that in no case velocity be allowed to increase beyond the maximum permissible velocity by selecting a smaller pipe size to dissipate the excess head.

6. The main advantage of LP-based method over the moving node method is the direct selection of pipes to be split into two sizes and their length. In this case, pipes 4, 6, 8, 10, 13, and 16 are of two sizes. As some of the pipes split into two sizes are not directly connected to the village reservoirs; selection of these pipes is a difficult task. Further, the moving node method allows the selection of smaller diameter pipes that violates the velocity criteria. It should be used carefully by the designer.

Annexure 6.5: Direct Pumping Main System (Shirpur)

A Case Study: Shirpur Water Supply Scheme

A pumped water transmission system from the clear water sump of the WTP to seven ESRs is shown in Figure 1. The objective is to design the pumping main with optimised HGL at the clear water sump of the water treatment plant (WTP). The system cost is optimised by considering both costs of pipes and the cost of energy. The cost of the pump is neglected in the optimisation process.

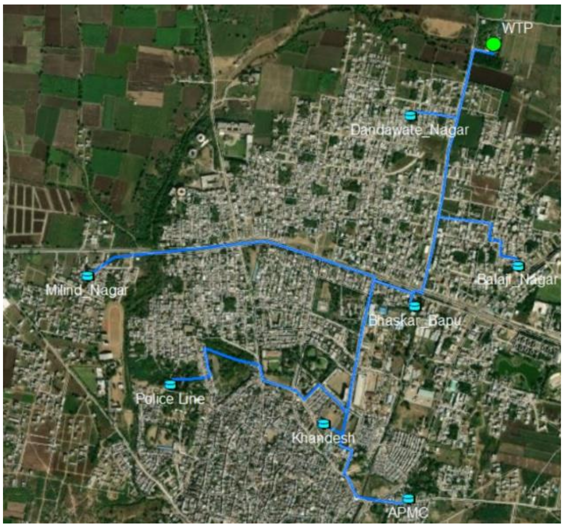


Figure 1: Network diagram showing various ESRs connected to WTP.

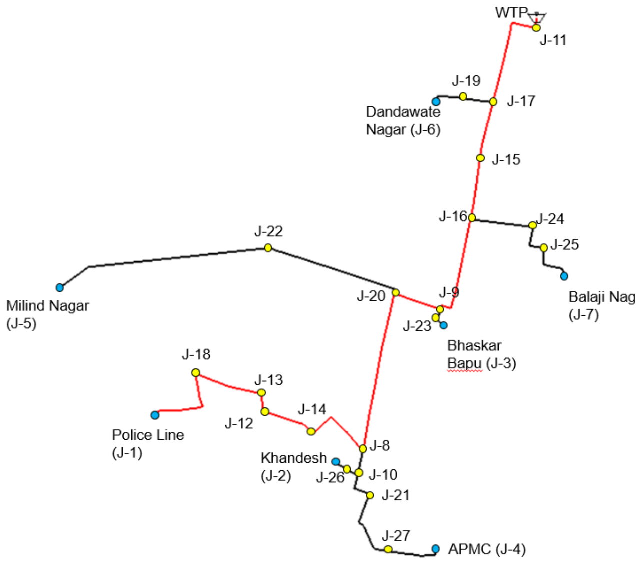


Figure 2: Network diagram on software

In Figure 2, the demand nodes (ESR nodes) are shown in blue colour. Other nodes are intermediate nodes (shown in yellow colour). Demands are given to the demand nodes only. The data is as under.

Design Data

1) Design period = 30 years
2) Hours of pumping in a day = 20
3) Cost of power = Rs. 7 per kWh
4) Pump efficiency = 70%
5) First stage period, n = 15 years
6) Interest rate, r = 8%
7) All pipe Materials = DI K-9
8) Hazen-Williams Coefficient for DI-K9 pipe = 140
9) Maximum velocity for a branch pipe connecting to service tank = 4.3 m/s
10) Minimum residual head at FSL of service tanks = 3 m

11) Minor head loss = 10% of frictional head loss

Pipe Cost: Pipe material, available pipe sizes, and cost per metre of length of pipe are as shown in Table 1.

Table 1: Pipe details

No.	Pipe Material	Diameter in mm	Cost (Rs/m length)
1	DI-K-9	80	750
2	DI-K-9	100	952
3	DI-K-9	150	1383
4	DI-K-9	200	1870
5	DI-K-9	250	2485
6	DI-K-9	300	3106
7	DI-K-9	350	3919
8	DI-K-9	400	4558
10	DI-K-9	450	5560
11	DI-K-9	500	6405
12	DI-K-9	600	8175
13	DI-K-9	700	10370

Flow Data: Flows at different stages are shown in Table 2.

Table 2: Flows at different stages.

Stage	Demand at 135 LPCD including losses (MLD)	Daily Demand at 135 LPCD and 15% losses (LPS)	Av discharge (LPS)	Daily demand in 20 hours (MLD)	Daily demand in 20 hours (LPS)	Av hours of pumping for Av discharge	Av hours of pumping for Av discharge
Initial	12	138.89		166.67	175.47		
Intermediate	17	196.76	167.82	236.11	248.58	212.02	17.06
Ultimate	23	266.20	231.48	319.44	336.31	292.44	17.39

Demands shown in Table 2 are inclusive of losses including minor losses.

Service tank data

Data of ESRs are shown in Table 3.

Table 3: ESR data

Operational Zone	Year: 2012	2027	2042	
	Demand (MLD)	Demand (MLD)	Demand (MLD)	Demand (LPS)
J-1(Police Line)	3.45	5	7.16	82.87
J-2) Khandesh)	3.35	4.71	6.22	71.99
J-3 (Bhaskar Bapu)	2.32	3.26	4.3	49.77
J-4 (APMC)	1.06	1.49	1.97	22.80
J-5 (Milind Nagar)	0.83	1.16	1.53	17.71
J-6 (Dandawate Nagar)	0.5	0.7	0.92	10.65
J-7 (Balaji Nagar)	0.49	0.68	0.9	10.42
Total	12	17	23	266.20

Putting values of rate of interest, r = 8%, for n=15 years,

Factor for first stage capitalisation = $\left\{\frac{1-(1+r)^{-n}}{r}\right\} = 8.559$ (Eq. a)

Factor for second stage capitalisation = $\frac{1}{(1+r)^{n}} = 0.315$ (Eq. b)

In the first stage, the pump requires less energy in comparison with the second stage. Hence, the discharge ratio and head loss ratio are computed which are shown in Table 4.

Table 4: Discharge ratio and head loss ratio

Ratio	Design Demand, Q_{des} (LPS)	Intermediate Demand (LPS)
1	2	3
	266.20	196.76
Discharge Ratio (Q/Q_{des})	1	0.73913043

Head Loss Ratio = $KQ^{1.852}$	1	0.571

Head loss is, hf = $KQ^{1.852}$, assuming all conditions same, the head loss ratio will be (discharge ratio)$^{1.852}$. The value of 0.571 is used in the computation of energy costs for the first period.

Optimisation of HGL at clear water sump of WTP involves two parameters - (i) optimisation of the capital cost of pipes and (ii) energy cost of pumping from WTP to various service tanks. It is required to find out the PW of combined pipe cost and energy for different HGLs of the pump at the clear water sump of WTP. The optimisation of HGL would occur for minimum PW of pipes and energy.

Find critical path: Critical path is the path having the least available frictional slope (= Available head difference/Length of the path) which is the WTP police line (shown in red colour) in Figure 2. The critical node is the one for which hydraulic slope is minimum (node police line). Calculations of the hydraulic slope are shown in Table 5.

Table 5: Calculations of hydraulic slope

Label	Full Supply Level, FSL (m)	Length of pipes between source to node (km)	Assumed Initial Source level (m)	Hydraulic slope (m/km)
Police Line	185	3.229	200	**4.6454**
Khandesh	183.5	2.283	200	7.2273
Bhaskar Bapu	163.26	1.365	200	26.916
APMC	185.5	2.785	200	5.2065
Milind Nagar	181.8	2.895	200	6.2867
Dandawate Nagar	189.5	0.67	200	15.672
Balaji Nagar	188.5	1.432	200	8.0307

(a) Deciding Initial Assumed Diameters of Pipe:

It is experienced that a transmission main, operates effectively for a velocity of 1 m/s. Consider one 1,000-metre length of the pipeline, considering DI pipe, after putting values in Hazen-Williams Equation,

$$V = \frac{0.849(140)D^{0.63}hf^{0.54}}{(4)^{0.63}(1000)^{0.54}} = 1.19052\ D^{0.63}hf^{0.54}$$

, or

$$hf / km = \frac{0.724V^{1.852}}{D^{1.16667}}$$ (Eq. c)

Now $Flow = \left(\frac{\pi D^2}{4}\right)V$ (Eq. d)

Equations (c) and (d) are used for making the assumption of the initial diameters of pipes.

For a flow of 23 MLD and velocity of 1 m/s, the diameter from Eq. (d) works out to 582 mm. Take one size above, i.e., 600 mm. Thus, the pipe size is assumed as 600 mm, which is for the first pipe (WTP to J-11) immediately from the source (clear water sump of WTP). The diameters of other pipes are assumed accordingly.

(b) Deciding Initially Assumed HGL: Again, for the known value of D = 0.582 m, and a value of V = 1 m/s, from Eq. (c), the value of

$$hf / km = \frac{0.724(1)^{1.852}}{(0.582)^{1.1667}} = 1.35 m / km$$

For this slope, Selected HGL at pump source node = FSL of critical demand node + minimum residual head (which is 3 m) required at FSL of the service tank + known value of computed slope in m/km × length in km of the critical path.

Thus, selected HGL at pump source node = 185 + 3 + 1.35 × 3.229 = 192.35 m, Say, 193 m.

PREPARATION OF HYDRAULIC MODEL

The network is solved using the established network software. A network line diagram showing various ESRs connected to WTP (Figure 2) has been used in the preparation of the hydraulic model. As GIS is used, values of the lengths of pipes are automatically scaled out.

As the total optimised cost is the sum of the capital cost of pipes and capitalised cost of energy, and as the total optimised cost is sensitive to energy cost, especially for the pipes on the critical path, a slightly liberal assumption is found to be more beneficial. The assumed pipe diameters and the node data are shown in Table 6.

Table 6: Details of network

Pipe Data						Junction Data			
Label	Start Node	Stop Node	Diameter (mm)	Length (m)		Label	J_ESR	Elevation (m)	Demand (ML/day)
1	2	3	4	5		6	7	8	9
P-1	J-3	J-23	500	25		J-1	Police Line	185	7.16
P-2	J-23	J-9	500	43		J-2	Khandesh	183.5	6.22
P-3	J-20	J-8	500	610		J-3	Bhaskar Bapu	188	4.3
P-4	J-15	J-16	500	307		J-4	APMC	185.5	1.97

P-5	J-16	J-9	500	398		J-5	Milind Nagar	181.8	1.53
P-6	J-11	J-17	500	401		J-6	Danda. Nagar	189.5	0.92
P-7	J-17	J-15	500	146		J-7	Balaji Nagar	188.5	0.9
P-8	J-9	J-20	500	187		J-8		164.29	0
P-9	WTP	J-11	600	45		J-9		164.85	0
P-10	J-12	J-13	300	74		J-10		164.15	0
P-11	J-14	J-12	300	210		J-11		161	0
P-12	J-8	J-14	300	274		J-12		160.48	0
P-13	J-13	J-18	300	253		J-13		160.65	0
P-14	J-18	J-1	300	324		J-14		162.34	0
P-15	J-10	J-26	300	75		J-15		166.2	0
P-16	J-26	J-2	300	19		J-16		165.61	0
P-17	J-16	J-24	300	215		J-17		164.82	0
P-18	J-24	J-25	300	174		J-18		162.63	0
P-19	J-25	J-7	300	143		J-19		167.5	0
P-20	J-17	J-19	300	133		J-20		163.7	0
P-21	J-19	J-6	300	91		J-21		163.25	0
P-22	J-20	J-22	300	1,217		J-22		161.22	0
P-23	J-22	J-5	300	132		J-23		163.98	0
P-24	J-8	J-10	350	94		J-24		165.85	0
P-25	J-21	J-10	300	110		J-25		166.04	0
P-26	J-21	J-27	300	355		J-26		159.03	0
P-27	J-27	J-4	300	127		J-27		156.8	0

LSL and FSL of service tanks are fixed considering the distribution network of concerned zones.

FSLs at city service tanks, nodes J-1 to J-7 are shown in Column 6 of Table 6 as these are the elevations of supply points to the reservoirs. For example, FSL at the J-1 node is 185 m and shown as elevation of feeder pipe to J-1 node. Nodal demands are given in Column 9 for all the reservoirs. These values are 0 at other junction nodes (shown in yellow colour in Figure 2) having no demand. Pipe lengths are given in Column 5 of Table 2.

1. Scenario of Source HGL=193 m

The required capacity of the pump needs to be computed to develop the required HGLs at the source. Consider Figure 3 in which various levels are shown for the assumed source HGL of 193 m.

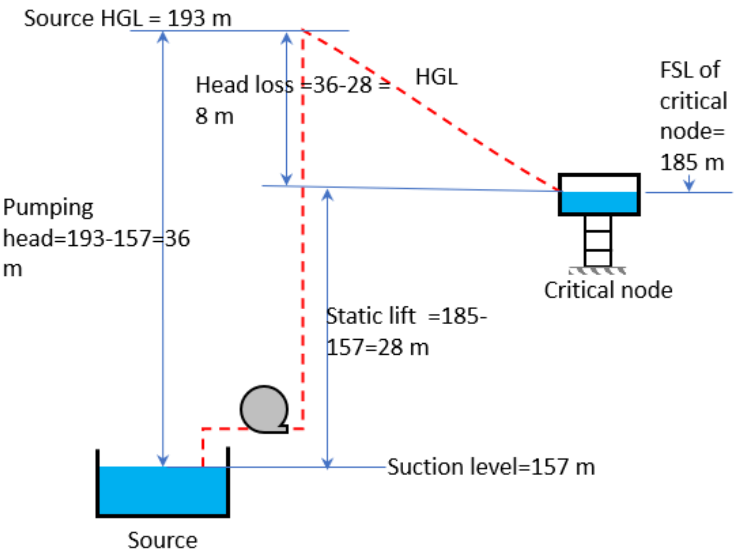


Figure 3: Network line diagram showing various ESRs connected to WTP.

The scenario of HGL of 193 m is created in the hydraulic model.

Initial Assumption: The diameters in the initial stage are assumed as considered in Table 6 and the hydraulic model is run. As the diameters are assumed in the first run, the diameters are not optimised.

The diameters and velocity are shown in Columns 5 and 6 of Table 7. The Residual Pressure (m) is shown in Column 4 of Table 8. It is observed that the residual pressures are less, and some of the values are even negative. Thus, it is required to increase the diameters in the initial stage.

Optimisation of Diameters of Pipes

Optimisation of diameters of pipe is an iterative method. It requires a hydraulic model prepared in any software. An iterative method is discussed below:

Table 7: Pipe data

Pipe Data				HGL = 193 m, Initial assumption of diameters		HGL = 193 m, Iteration-1		HGL = 193 m, Iteration-2		HGL = 193 m, Iteration-3	
Label	Start Node	Stop Node	Length (m)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)
1	2	3	4	5	6	7	8	9	10	11	12
P-21	J-19	J-6 (Dandawate Nagar)	91	400	0.085	400	**0.085**	200	0.339	200	0.339
P-20	J-17	J-19	133	350	0.111	350	**0.111**	200	0.339	200	0.339
P-22	J-20	J-22	707	250	0.361	250	0.361	300	0.251	200	0.564
P-23	J-22	J-5 (Milind Nagar)	641	250	0.361	250	0.361	200	0.564	150	1.002
P-17	J-16	J-24	215	150	0.589	300	**0.147**	250	0.212	200	0.332
P-18	J-24	J-25	174	150	0.589	250	**0.212**	200	0.332	200	0.332
P-8	J-9	J-20	187	600	0.691	600	0.691	500	0.995	500	0.995
P-25	J-21	J-10	110	200	0.726	300	0.323	300	0.323	200	0.726
P-5	J-16	J-9	398	600	0.867	700	0.637	600	0.867	600	0.867

Pipe Data				HGL = 193 m, Initial assumption of diameters		HGL = 193 m, Iteration-1		HGL = 193 m, Iteration-2		HGL = 193 m, Iteration-3	
Label	Start Node	Stop Node	Length (m)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)
P-4	J-15	J-16	307	600	0.904	700	0.664	600	0.904	600	0.904
P-7	J-17	J-15	146	600	0.904	700	0.664	600	0.904	600	0.904
P-3	J-20	J-8	610	500	0.905	500	0.905	500	0.905	500	0.905
P-6	J-11	J-17	401	600	0.942	700	0.692	700	0.692	700	0.692
P-9	28	J-11	43	600	0.942	700	0.692	700	0.692	700	0.692
P-24	J-8	J-10	94	350	0.985	400	0.754	500	0.483	300	1.341
P-10	J-12	J-13	74	300	1.172	300	1.172	300	1.172	350	0.861
P-11	J-14	J-12	210	300	1.172	400	0.659	450	0.521	350	0.861
P-12	J-8	J-14	274	300	1.172	400	0.659	500	0.422	400	0.659
P-13	J-13	J-18	253	300	1.172	300	1.172	400	0.659	350	0.861
P-14	J-18	J-1(Police Line)	324	300	1.172	300	1.172	300	1.172	300	1.172
P-26	J-21	J-27	288	150	1.29	250	0.464	300	0.323	200	0.726
P-27	J-27	J-4 (APMC)	195	150	1.29	150	1.29	300	0.323	200	0.726
P-19	J-25	J-7 (Balaji Nagar)	143	100	1.326	150	0.589	250	0.212	150	0.589
P-2	J-23	J-9	43	200	1.584	300	0.704	300	0.704	300	0.704

Pipe Data				HGL = 193 m, Initial assumption of diameters		HGL = 193 m, Iteration-1		HGL = 193 m, Iteration-2		HGL = 193 m, Iteration-3	
Label	Start Node	Stop Node	Length (m)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)	Diameter (mm)	Velocity (m/s)
P-1	J-3 (Bhaskar Bapu)	J-23	25	150	2.816	200	1.584	300	0.704	300	0.704
P-15	J-10	J-26	75	150	4.074	200	2.292	200	2.292	200	2.292
P-16	J-26	J-2 (Khandesh)	29	150	4.074	200	2.292	200	2.292	200	2.292

Iteration-1: With the modified diameters (Column 7), the hydraulic model is run. The diameters and velocity are shown in Columns 7 and 8 of Table 7. The Residual Pressure (m) is shown in Column 5 of Table 8. It is observed that the residual pressures are now above 3 m. Now we can go for a further optimisation process by decreasing the diameters for the pipes of which velocity values are low.

Table 8: Junction data

Data			Residual Pressure at FSL of service tanks (m)			
Label	Elevation (m)	Demand (ML/day)	After running model with initial assumption of diameters	Iteration-1	Iteration-2	Iteration-3
1	2	3	4	5	6	7
J-1(Police Line)	185	82.87	1.14	3.33	3.7	3.18
J-2 (Khandesh)	183.5	71.99	-2.4	5.46	4.89	4.46
J-3 (Bhaskar Bapu)	188	49.77	1.96	3.98	3.75	3.75
J-4 (APMC)	185.5	22.8	-0.49	3.41	4.97	3.21
J-5 (Milind Nagar)	181.8	17.71	8.9	9.65	8.53	4.34
J-6 (Dandawate Nagar)	189.5	10.65	2.96	3.24	3.11	3.11
J-7 (Balaji Nagar)	188.5	10.42	-0.04	3.61	3.57	3.16
J-8	164.29	0	26.3	27.04	26.39	26.39
J-9	164.85	0	26.68	27.43	26.96	26.96
J-10	164.15	0	26.21	27.06	26.49	26.06
J-11	161	0	31.88	31.91	31.91	31.91
J-12	160.48	0	28.19	30.37	29.99	29.54
J-13	160.65	0	27.73	29.92	29.53	29.24
J-14	162.34	0	27.16	28.72	28.25	28.07
J-15	166.2	0	26.07	26.43	26.34	26.34
J-16	165.61	0	26.32	26.86	26.6	26.6
J-17	164.82	0	27.6	27.87	27.87	27.87

Data			Residual Pressure at FSL of service tanks (m)			
Label	Elevation (m)	Demand (ML/day)	After running model with initial assumption of diameters	Iteration-1	Iteration-2	Iteration-3
1	2	3	4	5	6	7
J-18	162.63	0	24.75	26.94	27.31	26.79
J-19	167.5	0	24.92	25.2	25.12	25.12
J-20	163.7	0	27.71	28.45	27.8	27.8
J-21	163.25	0	26.82	27.92	27.35	26.67
J-22	161.22	0	29.79	30.53	30.12	29.12
J-23	163.98	0	27.07	28.22	27.75	27.75
J-24	165.85	0	25.55	26.6	26.31	26.23
J-25	166.04	0	24.93	26.37	26.02	25.93
J-26	159.03	0	24.61	30.52	29.95	29.52
J-27	156.8	0	30.21	34.11	33.69	32.36

Iteration-2: The diameters are decreased for the pipes whose velocities are low, the reduced diameters are shown in column 9 and increased velocity is shown in Column 10 of Table 7. The residual heads are shown in Column 6 of Table 8.

Subsequent Iterations: In Iteration 2, as the residual pressures in demand nodes (ESR nodes) are still more than 3 m, the diameters are further decreased for the pipes whose velocities are low and the process is repeated till it is not possible to reduce nodal residual heads, which is observed in the third iteration. Care should be taken that the pipe diameter is not less than 100 mm (or in exception case up to 80 mm).

The optimised diameters are shown in Column 11 of Table 7 and the computed values of velocities are shown in Column 12. The computed residual heads are shown in Column 7 of Table 8.

A summary of the pipe diameters and their cost is shown in Table 9.

Table 9: Summary of the pipe diameters and its cost

Iteration=1				Iteration=2				Iteration=3			
Diameter (mm)	Total Length (m)	Rate (Rs/m)	Cost (Rs)	Diameter (mm)	Total Length (m)	Rate (Rs/m)	Cost (Rs)	Diameter (mm)	Total Length (m)	Rate (Rs/m)	Cost (Rs)
150	338	1,383	467,454	200	1,143	1,870	2,137,410	150	784	1,383	1,084,272
200	129	1,870	241,230	250	358	2,485	889,630	200	2,017	1,870	3,771,790
250	1,810	2,485	4,497,850	300	1,766	3,106	5,485,196	300	486	3,106	1,509,516
300	1,019	3,106	3,165,014	400	253	4,558	1,153,174	350	537	3,919	2,104,503
350	133	3,919	521,227	450	210	5,560	1,167,600	400	274	4,558	1,248,892
400	669	4,558	3,049,302	500	1,165	6,405	7,461,825	500	797	6,405	5,104,785
500	610	6,405	3,907,050	600	851	8,175	6,956,925	600	851	8,175	6,956,925
600	187	8,175	1,528,725	700	444	10,370	4,604,280	700	444	10,370	4,604,280
700	1,295	10,370	13,429,150								
Grand Total	6,190		30,807,002	Grand Total	6,190		29,856,040	Grand Total	6,190		2,63,84,963

Optimised costs for the various HGL are computed as discussed above and are shown in Table 10. The above network is also optimised using JalTantra software. The comparison is shown in Table 10.

Table 10: Comparison of optimised cost for the various HGL

Source HGL (m)	Pipe cost by hydraulic model (Rs)	JalTantra-Pipe cost (Rs)	Percentage deviation (%)
193	2,63,84,963	2,75,40,737	-4.4
195	2,36,41,780	2,47,20,845	-4.6
197	2,23,55,529	2,30,84,267	-3.3
200	2,12,01,395	2,12,09,430	0.0
205	1,97,56,394	1,91,86,222	2.9
210	1,83,94,978	1,78,29,597	3.1

ENERGY COST

The energy costs are computed for both the stages - first and second period.

(A) Pump and Energy cost in First period (1 to 15 years)

Frictional head losses on the critical path are calculated and shown in Column 5 of Table 11.

Flow at the end of 15 years = Q1 = 17 MLD = 196.76 LPS

Energy cost (kWh) is computed by the formula:

$$kWh = \frac{QH}{102\eta}\left(\frac{24}{Pumping\ hours}\right) \qquad \text{(Eq. c)}$$

in which

Q = flow in LPS,

H = the total head (static head + frictional head loss) and pump efficiency which is 70%.

Table 11: Computation of total PW of pump and energy cost

Source HGL (m)	Lowest suction level (m)	Pumping head (m) (Col1-Col2)	Static lift (FSL of critical node-Lowest suction level) (m)	Head loss on critical path (m) (Col 3-Col4)	I Period (1 to 15 years)					II Period (15 to 30 years)			Total capitalised cost of pipe & energy (Rs) at the end of 15 years (Col 13 *Factor in Eq. b)	Total PW of pump & energy cost (Rs) (Col 10 + Col 14)
					Head loss on critical path for 1st period(m) (Col 5*0.571)	Total head for pump (m) (Col 4+Col 6)	Pump power, as per Eq. (c) (kWh)	Avg. annual energy cost (Rs)	Capitalised energy cost (Rs) (Col 9* Factor in Eq. a)	Pump power, as per Eq. (c) (kWh)	Avg. annual energy cost (Rs)	Capitalised energy cost (Rs) (Col 12*Factor in Eq. a)		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
193	157	36	28	8	4.568	32.568	107.7	46,97,282	4,02,06,286	161.1	71,61,765	6,13,00,972	1,93,24,623	5,95,30,909
195	157	38	28	10	5.71	33.71	111.5	48,61,993	4,16,16,123	170.0	75,59,641	6,47,06,582	2,03,98,213	6,20,14,337
197	157	40	28	12	6.852	34.852	115.3	50,26,703	4,30,25,961	179.0	79,57,516	6,81,12,192	2,14,71,803	6,44,97,764
200	157	43	28	15	8.565	36.565	120.9	52,73,769	4,51,40,716	192.4	85,54,330	7,32,20,606	2,30,82,189	6,82,22,905
205	157	48	28	20	11.42	39.42	130.4	56,85,546	4,86,65,309	214.8	95,49,020	8,17,34,630	2,57,66,164	7,44,31,473
210	157	53	28	25	14.275	42.275	139.8	60,97,323	5,21,89,902	237.1	1,05,43,709	9,02,48,654	2,84,50,139	8,06,40,042

2. Energy Cost

Using Eq. (c) the pump energy in kWh is computed and shown in Column 8 of Table 11.

Avg. annual energy cost is = kWh × Avg. hours of pumping for Avg. discharge × no. of days in a year × cost of electric energy

Values of average annual energy cost are shown in Column 9 of Table 11. We need to find out the capitalised cost of energy (Cf), which is given by the formula,

$$Cf = \text{CR} \left\{ \frac{1 - (1+\text{r})^{-\text{n}}}{\text{r}} \right\} \qquad \text{(Eq. d)}$$

In which CR is the annual energy cost.

For the value of r = 8% = 0.08 and n = 15 years, the value of the factor for the capitalised cost (or PW) of energy is computed which is,

$$Factor = \left\{ \frac{1 - (1+\text{r})^{-\text{n}}}{\text{r}} \right\} = \left\{ \frac{1 - (1+0.08)^{-15}}{0.08} \right\} = 8.559 \qquad \text{(Eq. e)}$$

$$\frac{1}{(1+\text{r})^{\text{n}}} = \frac{1}{(1+0.08)^{15}} = 0.315 \qquad \text{(Eq. f)}$$

Using Eq. (e), capitalised cost (or PW) of energy is computed which is shown in Column 10 of Table 11.

(B) Pump and Energy cost in Second Period (15 to 30 years)

Flow at the end of 30 years = 23 MLD = 266.2 LPS

Using Eq. (c), the pump energy in kWh is computed in Column 11 of Table 11.

Avg. annual energy cost is =kWh × Avg. hours of pumping for Avg. discharge × no. of days in a year × cost of electric energy

Avg. annual energy cost is computed and is shown in Column 12 of Table 11.

Now we need to find out the capitalised cost of energy at the end of the 15th year. Using Eq. (e) its values are computed and shown in Column 13 of Table 11.

The capitalised cost of pumps and energy cost at the initial (base) year = capitalised energy cost at the end of the 15th year)/ $\frac{1}{(1+\text{r})^{\text{n}}}$

Using this equation, the capitalised cost of pumps and energy cost in the current year are computed and are shown in Column 14 of Table 11.

The total PW of the pump and energy cost are shown in Column 15 of Table 11.

The values of the PW of total cost with respect to various HGL of the source are computed from the pipe cost (from Table 10) and total PW of energy cost (from Table 11) and its values are shown in Table 12.

Table 12: PW of total cost (Rs)

Source HGL (m)	Pipe cost (Rs)	Total PW of pump and energy cost (Rs)	PW of total cost (Rs)
193	2,63,84,963	5,95,30,909	8,59,15,872
195	2,36,41,780	6,20,14,337	8,56,56,117
197	2,23,55,529	6,44,97,764	8,68,53,293
200	2,12,01,395	6,82,22,905	8,94,24,300
205	1,97,56,394	7,44,31,473	9,41,87,867
210	1,83,94,978	8,06,40,042	9,90,35,020

The values of the present worth of total cost with respect to various HGL of the source are plotted in Figure 4.

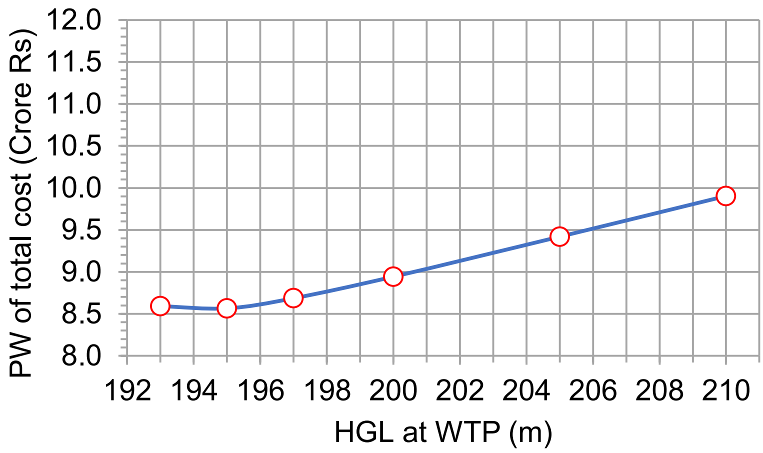


Figure 4: Present worth of total cost

Thus, it is concluded that the optimum HGL is 195 m.

Equalisation of heads at FSL of demand nodes: Though the pipe diameters are optimised, the heads at tanks are not equal. Hence, the moving node method as discussed in Section 5.10.2 has been adopted.

The norm of the residual head at FSL of the tank is 3 m. It is assumed that the equalisation of residual heads at tanks takes place if their values are in the range of 3 to 4 m.

Final Results: The final pipe results and the junction results with respect to source HGL of 195 m are shown in Table 13.

Table 13: Pipe and junction results

Pipe Results								Junction Results			
Label	Start Node	Stop Node	Length (m)	Diameter (mm)	Velocity (m/s)	Flow (Absolute) (L/s)		Label	Elevation (m)	Demand (L/s)	Pressure (m H2O)
1	2	3	4	5	6	7		8	9	10	11
P-1	J-3 (Bh. Bapu)	J-23	25	200	1.584	49.77		J-1(P. Line)	185	82.87	3.07
P-2	J-23	J-9	43	250	1.014	49.77		J-2 (Khandesh)	183.5	71.99	3.56
P-3	J-20	J-8	610	500	0.905	177.66		J-3 (Bh. Bapu)	188	49.77	3.11
P-4	J-15	J-16	307	500	1.302	255.56		J-4 (APMC)	185.5	22.8	3.85
P-5	J-16	J-9	398	500	1.248	245.14		J-5 (Mil. Nagar)	181.8	17.71	3.74
P-6	J-11	J-17	401	500	1.356	266.2		J-6 (D. Nagar)	189.5	10.65	3.96
P-7	J-17	J-15	146	500	1.302	255.56		J-7 (Bal. Nagar)	188.5	10.42	3.11
P-8	J-9	J-20	187	500	0.995	195.37		J-8	164.29	0	26.1
P-9	28	J-11	43	500	1.356	266.2		J-9	164.85	0	26.66
P-10	J-12	J-13	74	350	0.861	82.87		J-10	164.15	0	26.01
P-11	J-14	J-12	210	400	0.659	82.87		J-11	161	0	33.81
P-12	J-8	J-14	274	400	0.659	82.87		J-12	160.48	0	29.43

P-13	J-13	J-18	253	350	0.861	82.87		J-13	160.65	0	29.12
P-14	J-18	J-1(P. Line)	324	300	1.172	82.87		J-14	162.34	0	27.77
P-15	J-10	J-26	75	250	1.467	71.99		J-15	166.2	0	27.09
P-16	J-26	J-2 (Khandesh)	29	150	4.074	71.99		J-16	165.61	0	26.87
P-17	J-16	J-24	215	200	0.332	10.42		J-17	164.82	0	28.85
P-18	J-24	J-25	174	150	0.589	10.42		J-18	162.63	0	26.68
P-19	J-25	J-7 (Bal.Nagar)	143	150	0.589	10.42		J-19	167.5	0	26.15
P-20	J-17	J-19	133	250	0.217	10.65		J-20	163.7	0	27.5
P-21	J-19	J-6 (D.Nagar)	91	150	0.603	10.65		J-21	163.25	0	26.81
P-22	J-20	J-22	533	250	0.361	17.71		J-22	161.22	0	29.68
P-23	J-22	J-5 (Mil. Nagar)	815	150	1.002	17.71		J-23	163.98	0	27.36
P-24	J-8	J-10	94	350	0.985	94.79		J-24	165.85	0	26.5
P-25	J-21	J-10	110	250	0.464	22.8		J-25	166.04	0	25.88
P-26	J-21	J-27	288	250	0.464	22.8		J-26	159.03	0	30.56
P-27	J-27	J-4 (APMC)	195	200	0.726	22.8		J-27	156.8	0	33

It is observed from Column 6 of Table 13 that velocity in pipe P-16 (shown in red colour) is just nearing to a maximum velocity of 4.3 m/s. In such a situation, head dissipation may be done using an orifice plate/throttling valve.

The residual pressures at FSL of demand nodes as shown in Column 11 of Table 13 are in the range of 3 to 4 m. Hence, equalisation of pressures at the FSL of service tanks has occurred.

CONCLUSIONS

(a) Thus, it is concluded that the optimum HGL is 195 m.

(b) It is to be noted that all these residual heads are in the range of 3 to 4 m.

(d) Velocities in one pipe is 4.074 m/s, but it is less than 4.3 m/s. However, if any velocity is > 4.3 m/s, then such velocity shall be brought down in the range of 3 to 4 m/s by installing throttle valves.

(e) Capitalised cost of pipes and energy is Rs 23,641,780.

Annexure 6.6: A Case Study of Combined System

One gravity water supply scheme of a city (Figure 1) is considered for the sake of illustration. Water from the water treatment plant (WTP) is pumped to MBR through a pumping main of 500 mm diameter and a length of 5 km. MBR supplies water (Figure 1) by gravity to six ESRs, namely, A, EG, NG, NK, EH, and NH which are called demand nodes. These tanks supply water to their respective service areas to satisfy their water demands. There is one ZBR supplying water to six enroute villages.

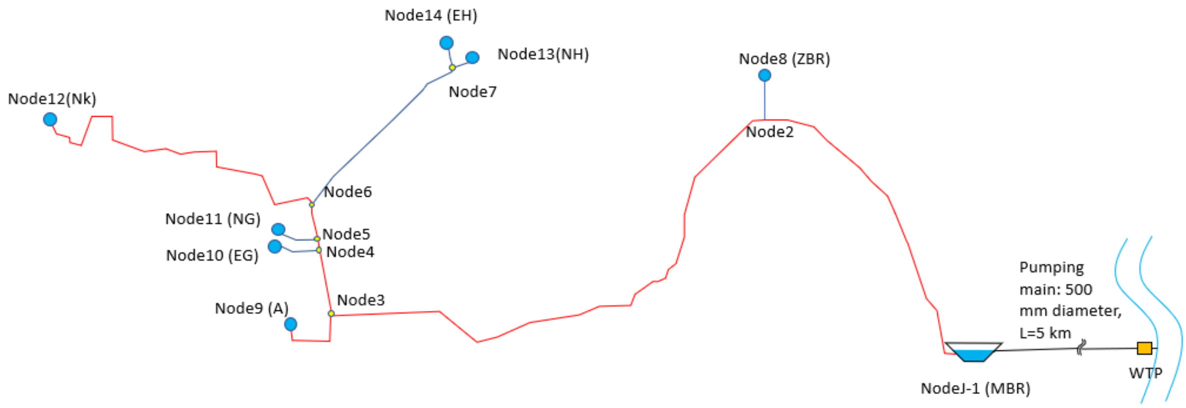


Figure 1: Transmission main

Flow Data: Flows at different stages are shown in Table 1.

Table 1: Flows at different stages

Year	Stage	Demand at 135 LPCD including 15% losses (MLD)	Daily Demand at 135 LPCD including 15% losses (LPS)	Ave discharge (LPS)	Av hours of pumping for Av discharge
2012	Initial	10	115.74		
2027	Intermediate	12	138.89	127.31	22.00
2042	Ultimate	15.44	178.70	158.80	21.33

Length of pumping main = 5 km, diameter = 500 mm.

Rate of interest = 8%, for n=15 years,

Factor for first stage capitalisation = $\left\{\frac{1-(1+r)^{-n}}{r}\right\} = 8.559$

Factor for second stage capitalisation = $\frac{1}{(1+r)^n} = 0.315$

The discharge ratio and head loss ratio are shown in Table 2.

Table 2: Discharge ratio and head loss ratio

Ratio	Design Demand, Q_{des} (LPS)	Intermediate Demand (LPS)
1	2	3
	178.70	138.89
Discharge Ratio (Q/Q_{des})	1	0.77720207
Head Loss Ratio = $KQ^{1.852}$	1	0.627

Optimisation of LSL of MBR involves two parameters - (i) optimisation of the capital cost of pipes and (ii) energy cost of pumping from WTP to MBR. It is required to find out the PW of combined pipe cost and energy for different LSLs of MBR. The optimisation of LSL of MBR would occur for minimum PW of pipes and energy.

Optimisation of Capital Cost of Pipes

JalTantra is a freeware software for the design and optimisation of Water Distribution Networks, developed by CSE IIT Bombay. An illustrative example as shown in Figure 1 is discussed.

General data (Table 3), pipe data, and node data (Table 4) are given to the online JalTantra software. To start with on the website of https://www.cse.iitb.ac.in/jaltantra/login.jsp, a user has to login and then feed the data of the network.

The nodes in JalTantra are shown in Figure 1. Initially, Source HGL is assumed as 55 m.

Table 3: General and commercial data

General data		Commercial Pipe Data		
Network Name	Example 5.6	Diameter (mm)	Roughness	Cost (Rs)
Organisation Name	ABC	80	140	750
Minimum Node Pressure (m)	3	100	140	952
Default Pipe Roughness 'C'	140	150	140	1,383
Minimum Head loss (m/km)	0	200	140	1,870
Maximum Head loss (m/km)	250	250	140	2,485
Maximum Water Speed		300	140	3,106

General data		Commercial Pipe Data		
Network Name	Example 5.6	Diameter (mm)	Roughness	Cost (Rs)
Maximum Pipe Pressure		350	140	3,919
Number of Supply Hours	24	400	140	4,558
Source Node ID	1	450	140	5,560
Source Node Name	Node1	500	140	6,405
Source Elevation (m)	50			
Source Head (m)	55			

Table 4: Node and pipe data

NODE DATA					PIPE DATA			
Node ID	Node Name	Elevation (m)	Demand (LPS)	Min. Pressure (3m)	Pipe ID	Start Node	End Node	Length (m)
2	node2	7.9		3	1	1	2	2,200
3	node3	11.39		3	2	2	3	7,123
4	node4	7.9		3	3	3	4	651
5	node5	7.9		3	4	4	5	15
6	node6	7.82		3	5	5	6	356
7	node7	9.97		3	6	6	7	1,859
8	node8	34	69.44	3	7	7	13	78
9	node9	25	12.59	3	8	7	14	63
10	node10	26	7.94	3	9	2	8	114
11	node11	26	23.36	3	10	3	9	660
12	node12	27	9.44	3	11	4	10	136
13	node13	26	37.22	3	12	5	11	104
14	node14	18	18.7	3	13	6	12	3348

After optimisation, the node results are shown in Table 5 and pipe results in Table 6.

Table 5: Node results

NODE RESULTS						
Node ID	Node Name	Demand (LPS)	Elevation (m)	Head (m)	Pressure (m)	Min. Pressure (m)
2	node2	0	7.9	46.50	38.60	3
3	node3	0	11.39	34.92	23.53	3
4	node4	0	7.9	34.08	26.18	3
5	node5	0	7.9	34.05	26.15	3
6	node6	0	7.82	33.14	25.32	3
7	node7	0	9.97	29.17	19.20	3
8	node8	69.44	34	37	3	3
9	node9	12.59	25	28	3	3
10	node10	7.94	26	29.72	3.72	3
11	node11	23.36	26	29	3	3
12	node12	9.44	27	30	3	3
13	node13	37.22	26	29	3	3
14	node14	18.7	18	21	3	3

Table 6: Pipe results

PIPE RESULTS										
Pipe ID	Start Node	End Node	Length (m)	Flow (LPS)	Speed (m/s)	Diameter (mm)	Roughness 'C'	Head loss (m)	Head loss (m/KM)	Cost (Rs)
1	1	2	1,972.61	178.69	1.42	400	140	7.98	4.04	8,991,174
1	1	2	227.39	178.69	1.12	450	140	0.52	2.28	1,264,267
2	2	3	7,123.00	109.25	0.87	400	140	11.58	1.63	32,466,634
3	3	4	651.00	96.66	0.77	400	140	0.84	1.30	2,967,258
4	4	5	15.00	88.72	0.92	350	140	0.03	2.12	58,785
5	5	6	356.00	65.36	0.92	300	140	0.91	2.55	1,105,736
6	6	7	154.55	55.92	1.14	250	140	0.72	4.64	384,062
6	6	7	1,704.45	55.92	0.79	300	140	3.25	1.91	5,294,016
7	7	13	78.00	37.22	0.76	250	140	0.17	2.18	193,830
8	7	14	16.44	18.7	2.38	100	140	0.87	52.89	15,650
8	7	14	46.56	18.7	3.72	80	140	7.30	156.80	34,921
9	2	8	114.00	69.44	3.93	150	140	9.50	83.38	157,662
10	3	9	209.91	12.59	1.60	100	140	5.34	25.42	199,830
10	3	9	450.09	12.59	0.71	150	140	1.59	3.53	622,481
11	4	10	136.00	7.94	1.58	80	140	4.36	32.09	102,000
12	5	11	56.64	23.36	2.97	100	140	4.52	79.86	53,925
12	5	11	47.36	23.36	1.32	150	140	0.53	11.09	65,494

13	6	12	919.07	9.44	0.53	150	140	1.90	2.07	1,271,070
13	6	12	2,428.93	9.44	0.30	200	140	1.24	0.51	4,542,104

The results are also obtained for other source HGLs, and the capital costs of the pipes is shown in Table 7.

Table 7: Capital costs of the pipes

Source HGL (m)	Pipe cost (Rs)
45	6,89,34,468
48	6,48,37,150
50	6,28,74,452
52	6,14,93,826
55	5,97,90,897
58	5,80,93,451
60	5,69,75,835

Energy Cost

For obtaining the optimum HGL, we need to find out the cost of energy required for pumping water from the clear water tank of WTP to the MBR. The clear water tank of WTP and MBR with HGL is shown in Figure 2. Calculations are shown in Table 8. The cost of MBR and pumps are not considered in the calculations.

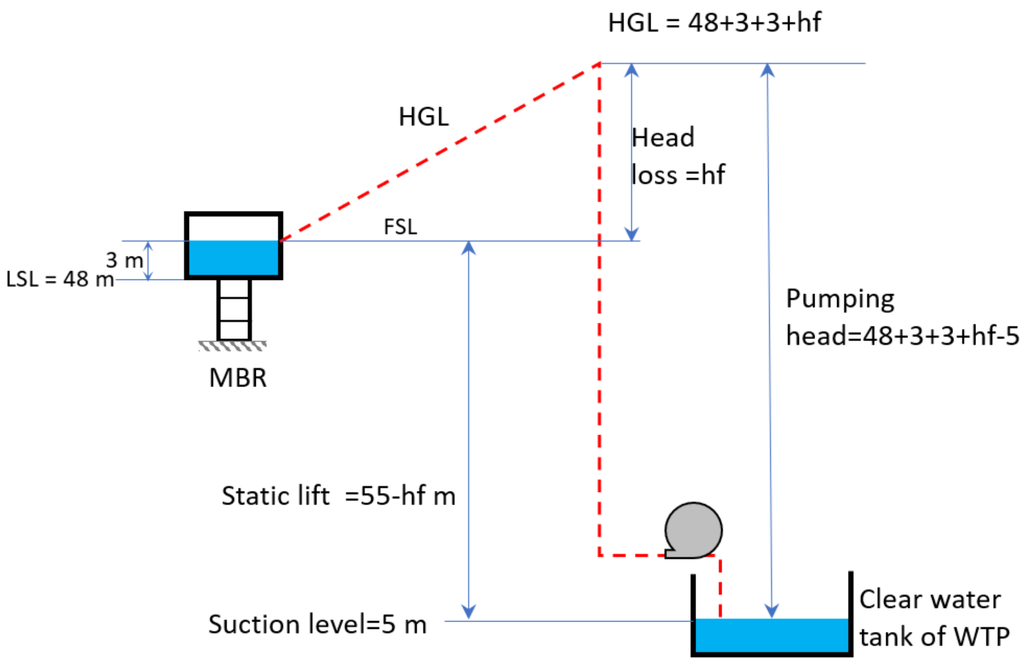


Figure 2: Pumping main

PW values of the total cost are plotted for different LSL of MBR in Figure 3. It is observed that the PW value of total cost is minimum for LSL of 48 m. Hence, the optimum LSL of MBR is 48 m.

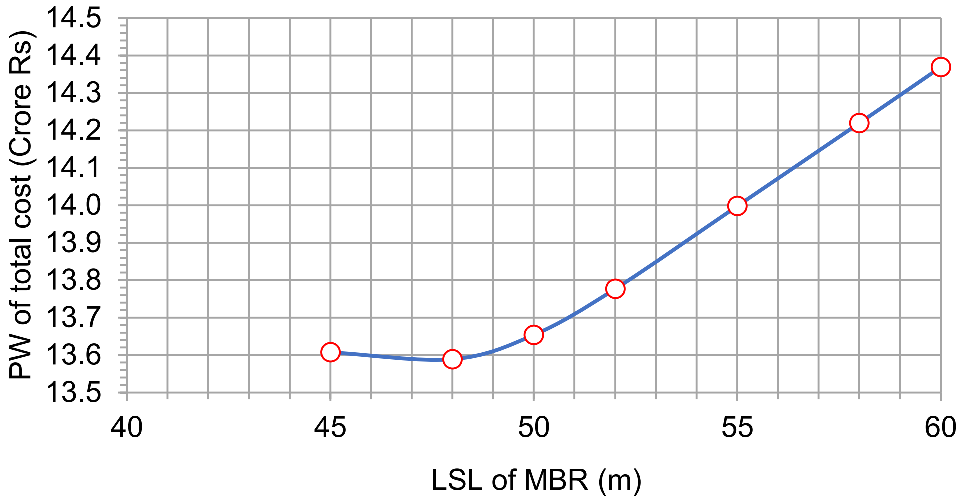


Figure 3: PW of total cost

Table 8: PW of energy charges

HGL required at MBR for feeding ESRs and ZBR(m)	Add 3 m depth of water and residual head at MBR 3 m	Lowest suction level (m)	Head loss including minor losses of 10%, hf (m)	Total head on pump for II period (HGL-Lowest suction level) (col 1-col 2) (m)	Static Lift(m)	I Period (1 to 15 years)					II Period (15 to 30 years)				Total PW energy c (Rs) (C 10 + C 14)
						Head loss for 1st period(m) (Col 4*0.627)	Total head for pump (m)	Pump power, as per Eq. (4) (kWh)	Avg. annual energy cost (Rs)	Capitalised energy cost (Rs) (Col 9*Factor in Eq6)	Pump power, as per Eq. (4) (kWh)	Avg. annual energy cost (Rs)	Capitalised energy cost at the end of 15 years (Rs) (Col 12*Factor in Eq6)	Total capital cost pump & energy (Rs) at the starting year (Col 13 *Factor in Eq 7)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
45	51	5	7.45	53.45	46.00	4.671	50.671	98.6	5,544,161	47,455,126	133.8	7,294,262	62,435,080	1,96,82,141	6,71,37,
48	54	5	7.45	56.45	49.00	4.671	53.671	104.4	5,872,408	50,264,754	141.3	7,703,676	65,939,452	2,07,86,865	7,10,51,
50	56	5	7.45	58.45	51.00	4.671	55.671	108.3	6,091,240	52,137,839	146.3	7,976,619	68,275,700	2,15,23,348	7,36,61,
52	58	5	7.45	60.45	53.00	4.671	57.671	112.2	6,310,072	54,010,925	151.3	8,249,562	70,611,948	2,22,59,831	7,62,70,
55	61	5	7.45	63.45	56.00	4.671	60.671	118.0	6,638,319	56,820,553	158.8	8,658,976	74,116,320	2,33,64,555	8,01,85,
58	64	5	7.45	66.45	59.00	4.671	63.671	123.9	6,966,567	59,630,180	166.3	9,068,390	77,620,693	2,44,69,280	8,40,99,
60	66	5	7.45	68.45	61.00	4.671	65.671	127.7	7,185,399	61,503,266	171.3	9,341,333	79,956,941	2,52,05,762	8,67,09,

Table 9: Present worth (PW) of total cost

Source HGL (m)	Pipe cost (Rs)	Total PW of pump & energy cost (Rs)	PW of total cost (Rs)	PW of total cost (Cr Rs)
45	68,934,468	67,137,267	136,071,735	13.61
48	64,837,150	71,051,619	135,888,769	13.59
50	62,874,452	73,661,187	136,535,639	13.65
52	61,493,826	76,270,756	137,764,582	13.78
55	59,790,897	80,185,108	139,976,005	14.00
58	58,093,451	84,099,460	142,192,911	14.22
60	56,975,835	86,709,028	143,684,863	14.37

Results for LSL = 48m

The node results and pipe results for optimum LSL of 48 m are shown in Tables 10 and 11.

Table 10: Node results

NODE RESULTS						
Node ID	Node Name	Demand (LPS)	Elevation (m)	Head (m)	Pressure (m)	Name of tank
2	node2	0	7.9	45.00	37.10	
3	node3	0	11.39	33.42	22.03	
4	node4	0	7.9	32.57	24.67	
5	node5	0	7.9	32.56	24.66	
6	node6	0	7.82	32.33	24.51	
7	node7	0	9.97	29.07	19.10	
8	node8	69.44	34	37	3	ZBR
9	node9	12.59	25	28	3	A
10	node10	7.94	26	29	3	EG
11	node11	23.36	26	29	3	NG
12	node12	9.44	27	30	3	NK
13	node13	37.22	26	29	3	NH
14	node14	18.7	18	21	3	EH

Table 11: Pipe results

Pipe ID	Start Node	End Node	Length (m)	Flow (LPS)	Speed (m/s)	Diameter (m)	Roughness 'C'	Head loss (m)	Head loss (m/KM)	Cost (Rs)
1	1	2	2,200.0	178.69	0.91	500	140	3.00	1.36	14,091,000
2	2	3	7,123.0	109.25	0.87	400	140	11.58	1.63	32,466,634
3	3	4	651.0	96.66	0.77	400	140	0.84	1.30	2,967,258
4	4	5	15.0	88.72	0.71	400	140	0.02	1.11	68,370
5	5	6	356.0	65.36	0.52	400	140	0.22	0.63	1,622,648
6	6	7	1,576.6	55.92	0.79	300	140	3.01	1.91	4,896,908
6	6	7	282.4	55.92	0.58	350	140	0.25	0.90	1,106,740
7	7	13	78.0	37.22	0.53	300	140	0.07	0.90	242,268
8	7	14	17.4	18.7	2.38	100	140	0.92	52.89	16,568
8	7	14	45.6	18.7	3.72	80	140	7.15	156.80	34,198
9	2	8	90.0	69.44	3.93	150	140	7.51	83.38	124,519
9	2	8	24.0	69.44	2.21	200	140	0.49	20.54	44,814
10	3	9	141.1	12.59	1.60	100	140	3.59	25.42	134,347
10	3	9	518.9	12.59	0.71	150	140	1.83	3.53	717,611
11	4	10	37.1	7.94	1.01	100	140	0.40	10.82	35,341
11	4	10	98.9	7.94	1.58	80	140	3.17	32.09	74,158
12	5	11	35.0	23.36	2.97	100	140	2.79	79.86	33,292
12	5	11	69.0	23.36	1.32	150	140	0.77	11.09	95,468

13	6	12	402.0	9.44	0.53	150	140	0.83	2.07	555,899
13	6	12	2,946.0	9.44	0.30	200	140	1.50	0.51	5,509,111

CONCLUSIONS

The following conclusions are drawn:

(a) The design of transmission main involves two parts - (i) optimisation of pipe diameters and (ii) equalisation of pressures. Optimisation can be made effectively using the freeware called “JalTantra”.

(b) For 24×7 water supply equalisation is most important. Without equalisation of pressures, there would be an inequitable distribution of water to the service tanks. After making equalisation of residual heads, the storage tanks receive water just equal to their design requirement. Hence, without equalisation design of the transmission main is incomplete.

(c) The JalTantra software is most effective in optimising diameters as well as making equalisation of residual pressures.

(d) By bringing residual head to 3 m for all ESRs/ GSRs, the storage tanks receive water just equal to their design requirement.

(e) The maximum velocity in the pipe network is less than 3.93 m/s which is less than 4.3 m/s.

**

Annexure 6.7: Break Pressure Tank

Illustrative Example: To illustrate the application of the criteria developed, data on the water supply scheme of one city is considered.

Data: The gravity raw water main is from BPT to the aeration fountain. The gravity main is a pre-stressed concrete pipeline with of length 30,900 m and an internal diameter of 1,500 mm. The gravity main is in form of an inverted siphon as shown in Figure 6.5. Even after the stoppage of pumps, the pipeline remains full up to 396.0 m RL of aeration fountain.

The outlet of gravity main (elevation of destination) is in the form of a lip of aeration fountain kept at RL of 396 m. Computation is shown in Table 1.

Table 1: Computation of sizes of BPT

S. No.	Parameter	Formula	Immediate Stage	Ultimate Stage
1	*D = Diameter of pipe (mm)*	Data	1500	1500
2	L = Length of d/s gravity pipeline (m)	Data	30,900	30,900
3	Elv_D = Elevation of destination (Lip of aerator)	Data	396	396
4	A = Cross-sectional area of gravity pipe, (m2)	$A = \frac{\pi}{4} D^2$	1.767	1.767
5	HW C-value	Data	145	140
6	Q_{MLD} = Flow (MLD), Q_{MLD}	Data	110	150
7	Q = Flow (m^3/s)	$Q = \frac{Q_{MLD} * 1000}{(24 * 3600)}$	1.273	1.736
8	V_0 = Velocity in gravity pipe (m/s)	V_0=Q/A	0.720	0.982
9	hf = Frictional Head loss (m)	$hf = \frac{10.67L}{\left(C^{1.852} D^{4.87}\right)} Q^{1.852}$	7.111	13.478
10	H_f = Total Frictional Head loss + minor losses (m)	Hf = 1.1 (hf)	7.822	14.826
11	LSL of BPT	LSL of BPT = ElvD + Hf (of immediate stage)	396+7.822= 403.822 (Rounded 403.8)	

S. No.	Parameter	Formula	Immediate Stage	Ultimate Stage
12	FSL of BPT	FSL of BPT = ElvD + Hf (of ultimate stage)		396+14.826 = 410.826 (Rounded 410.9)
13	Bottom RL of BPT	LSL-0.5	403.822-0.5 = 403.322	
14	Top RL of BPT			410.826+3.5 = 414.3 m
15	F = Friction constant	$F = \frac{Hf}{(V_0)^2}$	15.070	15.361
16	A_T = Cross-sectional area of BPT(m^2)	$A_T = \frac{4AL}{F^2 V_0^2 g}$	188.872	97.767
17	D_{BPT}= Diameter of BPT	$D_{BPT} = \left\{\frac{4A_T}{(\pi)}\right\}^{0.5}$	15.51 (Selected 10.0 m)	11.16 (Selected 10.0 m by reducing area by about 25%)
18	Volume of BPT	A_T × (FSL-Bottom RL)	550.10	550.10
19	Retention time (minutes)	Volume/Q	7.20	5.28

Conclusions:

i) From above Table A-1, the diameter of BPT is 10.0 m and retention time is 7.20 minutes in the immediate stage and 5.28 minutes in the ultimate stage.

Thus, though the length of gravity main is about 31 km and the pipe diameter is large i.e., 1500 mm, a very small BPT is adequate though, the downstream pipeline remains practically filled under no-flow conditions.

ii) Even though the BPT area, as per S. No.16, is reduced from 188.87 m^2 to 78.54 m^2, i.e., a reduction by 58.40%, the maximum WL attained in the immediate stage is 406.31 m as against steady state WL of 403.82 m (S. No.13). This shows that overshoot above steady state WL is marginal and much below FSL.

iii) The diameter of 11.16 m calculated for the ultimate stage is reduced to 10.0 m, thus reducing the cross-sectional area by 24.5%. On computation on basis of equations, WL rise above steady state WL (410.82 m) is just 0.10 m and thus very marginal compared to a safe margin of 3.0 m to be provided above FSL.

***

Annexure 6.8: Design of Thrust Blocks

To design a thrust block for 900 mm diameter main conveying water at 11 kgs/cm^2 pressure (P).

The deviation angle α is 45° and density of concrete is 2,300 kgs/m^3. Soil density is assumed to be 1800 kgs/m^3 and angle of internal friction $\phi = 30^0$.

Assume minimum cover of earth is 600 mm. Cohesion is 0 for sandy soils.

Horizontal Thrust: $\mathrm{F} = 2\ \mathrm{pA}\ \mathrm{Sin}\ \alpha/2$

Cross-sectional area $\mathrm{A} = \left(\frac{\pi}{4}\right)(90)^2 = 6364\ \mathrm{sq.cms.}$

$\mathrm{Sin}\ \alpha/2 = 0.382$

$\mathrm{F} = 2\ (11)\ (6364)\ (0.382)\ (10^3) = \ 53.48\ \mathrm{tonnes}$

(i) Lateral resistance to counteract the horizontal thrust:

Try a thrust block of size = 3.2M × 3.2M × 3.2M

Weight of thrust block = 3.2M × 3.2M × 3.2 M × 2.3 = 75.36 tonnes

Weight of water in the pipe = 0.785 × $(0.9)^2$ × 1 × 3.2 = 2.03 tonnes

Weight of earth = 0.9 × (3.2) (0.6) (1.8) = 3.11 tonnes

Total Weight 80.50 tonnes

Total force available considering frictional resistance of soil = 80.5 (0.3) = **24.15 tonnes**

(ii) Lateral resistance of soil against the block:

$$f_p = \gamma_s \frac{(\mathrm{H})^2}{2} \cdot L \cdot \frac{1 + Sin\theta}{1 - Sin\theta} + 2CHL\sqrt{\frac{1 + Sin\theta}{1 - Sin\theta}}$$

By assuming cohesion as 0, the above equation

$$\mathrm{Yields} = 1.8\frac{(3.2)^2}{2}\ (3.2)\ \frac{(1.5)}{0.5} = 88.47\ tonnes$$

(iii) Lateral resistance of soil when the thrust block is free to yield away from the soil mass i.e., the portion of projected pipes -

$$f_a = \gamma_s\ h\ \frac{1-Sin\theta}{1+Sin\theta} - 2C\sqrt{\frac{1-Sin\theta}{1+Sin\theta}}$$

$$= (1.8)(0.9)\frac{(0.5)}{1.5} = 0.54\ \mathrm{tonnes}$$

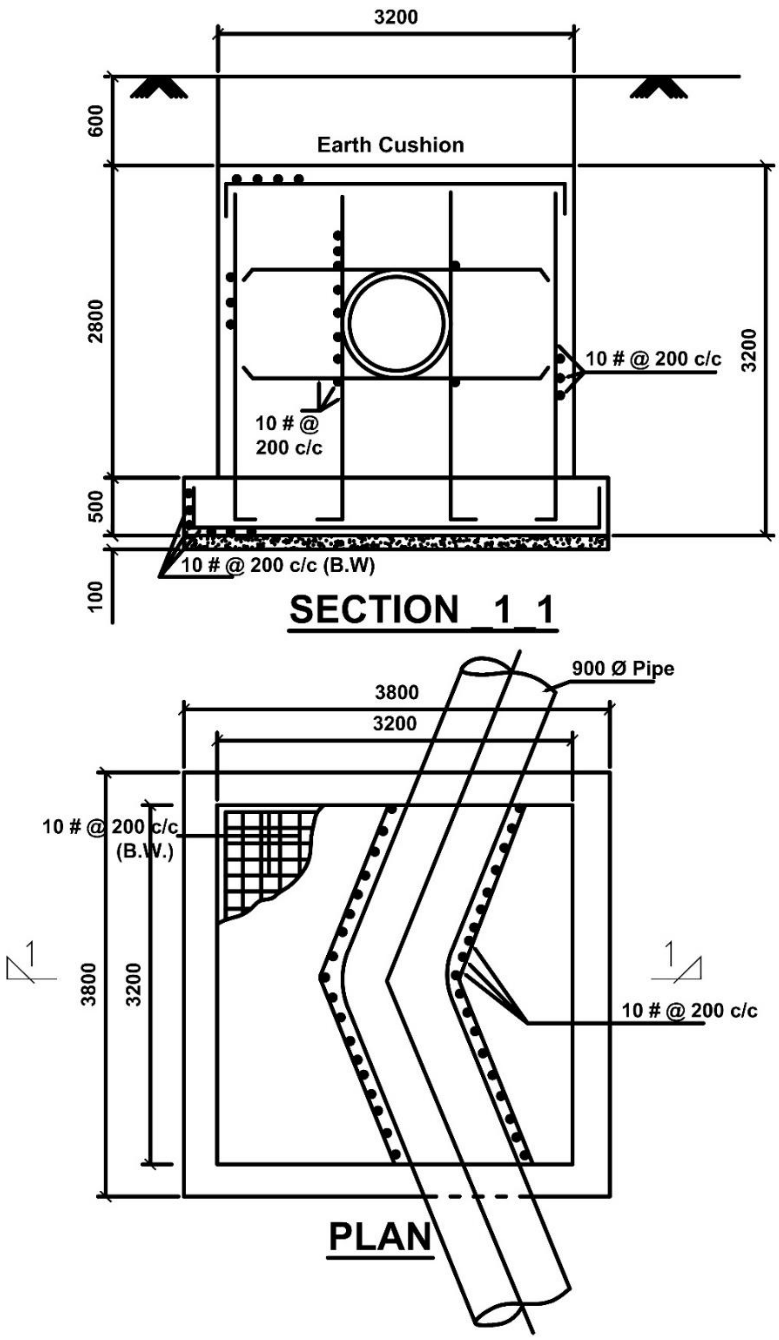
Total lateral resistance = 24.15 + 88.47 + 0.54 = **113.16 tonnes/m^2**

Total horizontal thrust = **53.48 tonnes**

Factor of safety = 113.16/53.48 = 2.19 which is O.K.

Reinforcement:

The minimum surface reinforcement in all thrust blocks shall be 5 kgs/sq. m (as per IRG 21-1972 Article 306.4). The spacing of these bars is not to exceed 500 mm. Hence provide 10 bars at 200 c/c which is more than 5 kgs/sqm.



Thrust Block for 45° Horizontal Bend

Annexure 6.9: Design of Air Vessel

DATA

Discharge through pipe line	= 1.944 cumecs
Material of pipe line	= Steel
Diameter of pipe line	= 1550 mm
Thickness of pipe (ct)	= 10 mm
Static Head	= 120 mm
Length of pumping main	= 18000 m
Total Head including friction Head & other losses of 15 metre.	= 135 m
Design head of pumps.	= 150 m
Atmospheric Head	= 10 m

H_0 = Absolute Head = Total head + Atmospheric Head = 135 +10 =145 m

V_0 = Initial velocity $= \frac{1.994}{\left(\frac{\pi}{4}\right)(1.55)^2} = 1.03\ \mathrm{m/sec}$

C = Water Hammer Wave velocity,

$$= \frac{1425}{\sqrt{1+\frac{kd}{E_{ct}}}}$$

$$= \frac{1425}{\sqrt{1+[\frac{2.07x10^8}{2.1x10^{10}x0.01}]}} = 896.27\ m/s$$

Water Hammer Head $= \frac{cV_0}{g} = \frac{896.27 \times 1.03}{9.81}$

= 94.10 metres

Pipe Parameter $\rho = \frac{cV_0}{2gH_0}$

$$= \frac{896.27 \times 1.03}{2 \times 9.81 \times 145}$$

= 0.324

2ρ = 0.65

AIR VESSEL PARAMETER = $2C_0C/Q_0L$

Referring Chart f_0, k_c as 0.50 for limiting upsurge to 1.20 H_0 and down surge to 0.5 H_0

Air vessel parameter for 2ρ = 0.65 is calculated as follows:

From chart for 2ρ = $1.00 \times (2C_0C/Q_0L) = 10.50$

For 2ρ = 0.5, $2C_0C/Q_0L$ = 6.50

By interpolation for 2ρ = 0.65 and for k_c = 0.5

Air vessel parameter $2C_0C/Q_0L$ = 7.70

Volume of air C_0 = $\frac{7.7 \times 1.944 \times 18000}{2 \times 896.27}$

= 150.31 Cubic metres

Volume of Air Vessel =$C_0[H_0/H_{min}]^{1/1.20}$

= $150.31[(145)/0.5 \times (145)^{1/1.20}]$

= $150.31 \times (2)^{1/1.20}$

= 267.20 Cum

Increase the capacity by 20% to cater for upsurge of 1.20 H_0

= 267.20 × 1.2

= 320 m^3

WATER COLUMN SEPERATION LENGTH

The water column separation is calculated on the basis of the following formula:

$$_1{}^2 - V_2{}^2 = (2g/L)\{(t_1 - t_2)\,V_1\,\{H + F\,(V_1{}^2/V_0)\}]$$

H = Static Head, (Absolute Head)

F = Loss of head due to friction

V_1, V_2 = Velocities at instances t_1 and t_2.

(t_1-t_2) = Period between time intervals in seconds.

V_0 = Initial Velocity.

L = Length of pipeline

Initial velocity will come to rest over a time period after the stoppage of pumps. Assuming a time interval of 0.20 seconds and by using above formula, the subsequent velocities are calculated till the final velocity (V_n) is almost Zero. The water column separation length I is given by Laws =

Iwcs Σ [V_1 + V_2 + ……… V_n] (t_2-t_1)

For the given diameter of pipe and for the calculated water column separation Length the volume of water required to be stored in Air vessel is calculated.

For Worked Example

$$(1.03)^2 - V_2{}^2 = 2 \times \frac{9.81}{18000}(0.20)(1.03)[145 + 15\frac{(1.03)^2}{1.03}]$$

$$(1.01)^2 - V_3{}^2 = 2 \times \frac{9.81}{18000}(0.20)(1.01)[145 + 15\frac{(1.03)^2}{1.01}]$$

Repeat n times till V_n = 0.01m /sec

Then $V_1+V_2+V_3+$..............V_n (0.20) = say = 6.1 metres.

For a pipe of 1.55 per dia volume of water required to fill this separation length

$$\frac{\pi}{4}(1.55)^2(6.10) = 11.51\text{Cum}$$

FIXING THE SIZE OF VESSEL AND LEVELS OF WATER AND AIR IN AIRVESSEL CHAMBER

(i) Air And Water Volume

Air Vessel volume required = 320 Cum

If two vessels are provided volume of each vessel = 160 Cum

Provide 90 Cum of Air and 70 Cum of water in each vessel.

(ii) Determination Of Size of Air Vessel

Absolute Head at working head of pumps 150+ 10.35 160.35 metres.

Maximum upsurge permitted 160.35 × 1.2 = 192.42 metres

Pressure 19.25 kg/cm³

Using 25 mm thick MS Plate i.e., 22 mm+3mm for corrosion allowance

$$d = \frac{2f_t \times e \times t}{p}$$

f_t = Permissible tensile strength in steel plates = 1260 kgs/cm²

e = Weld efficiency say 0.9

t = Thickness in cms of plate = 2.2 cm

P =Pressure in kg/cm²

$= \frac{2 \times 1260 \times 1.90 \times 2.20}{19.25}$

= 259.20 cms

= Say 260 cms

Provide 2.60 m dia of vessel with a length L and two hemispherical ends.

Volume of (two hemispheres) spherical portion $=\frac{4}{3} \times \pi(1.3)^3 = 9.2$ Cum.

Total Volume of cylinder = 160 cum - 9.20 = 150.80 cum

Length of vessel of 2.6 m dia with volume 150.80 cum is = 28.40 metres

Provide 2 vessels each of 2.6 m dia and 28.40 m long with hemispherical ends.

(iii) Fixing of Levels of Water and Air in the Vessel

The levels are fixed by trial by assuming a depth and calculating volume in cylindrical and spherical portions,

(a) Normal Working Level

Volume of Air = 90 cum

Volume of Water = 70 cum

The normal working level is fixed by trial by assuming 1.15 metre of water by depth from bottom. Volume of water = 70.95 Cum which is more than required 70 Cum. Hence normal working level will be at 1.15 m from bottom of vessel.

(b) Upper Emergency Level

Air dissolves in water in the vessel. Assuming that 10% air dissolves in water the level of water rises by 10% of volume of air, i.e.:

Volume of water = 70 Cum + 10% of 90 Cum = 79 Cum.

The depth of water from bottom will be 1.35 m which gives volume of water as 79 Cum.

Hence upper emergency level will be 1.35 m from bottom of vessel.

(c) Lower Emergency Level

When pumps trip as per water column separation about 11.51 Cum of water is required to fill the pipeline. As calculated volume of water at a depth of 1.00 m from bottom of vessel = 56.43 Cum. Volume of water at normal working level is 71 Cum.

Quantity of water available is the difference between normal working level and lower emergency level.

Annexure 8.1: Sizing of Units of Conventional WTP

Design Basis		
Flow rates:		
a.	Overall proposed water demand (Including 3% losses in sludge and backwash water)	: 20 MLD (22 hr. basis)
b.	Design capacity of Water Treatment Plant based	: (20 MLD / 22 hr.) × 24 = 21.81 MLD
	On 24 hr. basis	Say 22.00 MLD
c.	Corresponding flow rate	: 916.67 cum / hr.
		Say 920 cum / hr.
d.	TSS in raw water	: 100 mg/L
1	**Cascade Aerator**	
1.a	No of units	: 1, one
1.b	Flow rate	: 920.00 cum/hr.
1.c	Area loading	: 0.030 sqm/cum/hr.
1.d	Area required	: 920 × 0.030 = 27.62 sqm
1.e	Diameter of central shaft (RCC)	: 0.90 m ID, (Velo. 0.41 m/sec)
	(ID –Internal Diameter, OD –Outer Diameter)	: 1.20 m OD (0.90 + 0.15 + 0.15)
1.f	Area of central shaft	: (3.14 / 4) × 1.20^2 = 1.13 sqm
1.g	Total area required	: 27.62 + 1.13 = 28.75 sqm
1.h	No of cascades	: 4, Four
1.i	Let tread of cascade be	: 0.65 m
1.j	Diameter of lowest cascade	: (0.65 × 8) + 1.20 m = 6.40 m
1.k	Actual area provided	: (3.14 / 4) × 6.40^2 = 32.15 sqm
		: (hence OK)
1.l	Rise of cascade	: 0.20 m
1.m	Total drop over cascade	(4 + 1) × 0.20 = 1.00 m
1.n	**Collection launder**	: Two semi-circular channels
1.n.1	Flow per channel	: 920 / 2
		: 460 cum/hr.
1.n.2	Width	: 0.90 m
1.n.3	Velocity	: 0.60 m/sec
1.n.4	SWD (Side Water Depth)	: 460 / (3600 × 0.90 × 0.60)
		: 0.24 m, say 0.35 m
1.n.5	Freeboard	: 0.30 m
2	**Elevated channel with Parshall flume**	
2.a	No of units	: 1, one

2.b	Flow rate	: 920 cum/hr (0.26 cum/sec)
2.c	Width of channel	: 1.25 m
2.d	Width of Parshall flume	: 0.30 m, (1.0 foot)
	Design of Parshall Flume (Ref IS14371:1996, Parshall Flume)	
2.e	Head U/S of Flume (Table 3, Page 6)	
	For throat width of 0.3 m, Q = 0.679 ha^1.521	
	Q= 0.25 cum/sec	
	Ha = 0.52 m	
	Depth of water U/S (Ha + N) = 0.52 + 0.238 = 0.76 m, say 0.8 m	
	Head range, Ha = 0.03 m to 0.75 m, hence OK	
	Discharge range, cum/sec × 10^3: 250, (3.5 to 400) hence OK	
2.f	Approach Channel, Froud No. (5.2.2., Page No. 2)	
	Fr = Q / [A × sqrt (g × h)]	
	Q = 0.26 cum/sec,	
	A = c/s area U/S of flume: 1.25 m × 0.80 m swd = 1.0 sqm	
	g = 9.81 m/sec^2	
	h = depth of water upstream of flume = 0.80 m	
	Hence Fr = 0.25 / [1.0 × sqrt (9.81 × 0.80)]	
	= 0.09 (less than 0.5)	
2.g	Velocity in channel (upstream of flume)	: V = Q / A
		V = 920 / (3600 × 1.25 × 0.80)
		: 0. 26 m/sec
2.h	Conditions on d/s of flume	: Free-falling free discharge
2.j	Free board	: 0.30 m on upstream side of flume
3	**Flash mixer**	
3.a	No of units	: 1, One
3.b	Flow rate	: 920 cum/hr.
3.c	Let Size be	: 2.25 m × 2.25 m × 3.50 m swd
3.d	Detention time	: [(2.25 × 2.25 × 3.5) / 920] × 60 = 1.15 min. (More than 1 min, hence OK)
3.e	Rapid agitator	: Radial turbine type mixer
3.f	Velocity gradient (G)	: 300 m/sec/m
3.g	Inlet chamber	: 2.25 m × 1.25 m
3.h	Outlet chamber	: 2.25 m × 1.25 m
3.i	Diameter of inlet pipe to clarifier	: 0.60 m (600 mm)
3.j	Velocity in pipe	: 0.90 m/sec (hence OK)
4	**Clariflocculator**	
4.a	No of units proposed	: 1, one
4.b	Capacity / Flow per unit	: 920 cum/hr
4.c	Diameter of central shaft	: 0.90 m (ID), 1.30 m (OD)

	(ID –Internal Dia, OD –Outer Dia.)	(1.30 = 0.90 + 0.20 + 0.20)
4.d	Velocity through shaft	: 0.40 m / sec
4.e	Size of outlet openings	: 0.60 m × 0.30 m × 4 no
4.f	Velocity through openings	: 0.35 m / sec
4.g	Let Diameter of flocculator be	: 13.00 m (ID), 13.40 m (OD)
		(OD =13.00 + 0.20 + 0.20)
4.h	Side water depth	: 3.50 m
4.i	Volumetric capacity	: 3.50 × 3.14/4 × (13.00^ ² - 1.30 ^ ²)
		: 461.3 cum
4.j	Detention time	: (461.3/920.) × 60 = 30.05 minute
4.k	Let Overall dia of clariflocculator be	: 33.00 m (Internal Diam.)
4.l	Side water depth	: 3.50 m, F.B. 0. 50 m
4.m	Volumetric capacity	: 3.5 × 3.14/4 (33.0^ ² - 13.40^ ²)
		: 2,500 Cum
4.n	Detention time	: (2,500/920) = 2.71 Hr.
4.o	Surface loading	: 920 / [3.14/4 (33.0^ ² - 13.40^ ²)]
		: 1.29 cum/sqm/hr (31 cum/sqm/day)
		Hence OK.
4.p	Actual weir length	: (3.14 × 33.0) = 103.7
4.q	Weir loading	: (3.14 × 33.0) = 103.7 rm
		: (920/103.7) × 24
		: 213 cum/rm/day
		Hence OK
4.r	Size of Outlet openings from flocculator to clarification zone: (At bottom)	
4.r.1	Velocity through opening	: 0.005 m/sec
4.r.2	Area of opening	: (920/3,600) / 0.005
		= 51.11 sqm
4.r.3	Perimeter of flocculation wall	: 14.40 × 3.14 = 45.21 m
4.r.4	Height of opening	: 51.11 / 45.21 = 1.13 m,
		Say 1.15 m
	(Flocculator wall is supported on columns. If eight columns are provided, the size of each opening is approximately 45.21 / 8 = 5.61 m × 1.15 m)	
4.s	**Collection launder**	: Two semi-circular channels
4.s.1	Flow per channel	: 920 / 2
		: 460 cum/hr.
4.s.2	Width	: 0.75 m
4.s.3	Velocity	: 0.60 m/sec
4.s.4	SWD (Side Water Depth)	: 460 / (3,600 × 0.75 × 0.60)
		: 0.28 m, say 0.40 m

5	**Rapid sand gravity filter beds (four beds): Constant Rate Filtration with Influent Splitting weirs**	
5.a	No of beds proposed	: 4, four
5.b	Capacity / Flow rate	: 920 cum / hr.
5.c	Flow rate per bed	: 920 / 4 = 230 cum / hr.
5.d	Configuration of bed	: Two sections with central gutter (gullet)
5.e	Let Net area of filtration/bed	: 6.0 m × 4.2 m each section × 2
		: 25.20 × 2 = 50.4 sq. m.
5.f	Rate of filtration	: 230 / 50.4 = 4.56 cum/sqm./hr.
		(less than 5 cum/sqm/hr., hence OK)
5.g	Width of central gutter	: 0.60 m I.D. + 0.20 m + 0.20 m = 1.00 m
		(Wall thickness 0.20 m)
5.g.1	Width of filter unit	: 4.2 m + 1.00 m + 4.2 m = 9.40 m
5.g.2	Overall size of bed	: 9.40 m × 6.0 m × 3.65 m SWD
	Ratio of length to width	: 9.40 / 6.0 = 1.56, hence OK
5.h	Depth of filter bed	
5.h.1	Depth of supporting gravel	: 0.60 m (size 2 mm to 50 mm, graded)
5.h.2	Depth of filter silica sand	: 0.75 m (E.S. 0.55 mm, U.C. 1.5 to 1.7)
5.h.3	Depth of water column over media	: 2.30 m
5.h.4	Total depth	: 3.65 m
5.h.5	Free board	: 0.50 m
5.i	Backwash	: Separate wash of both sections
5.i.1	Backwash flow rate	: 36 cum/sq. m/hr.
5.i.2	Flow rate	: 25.20 sq. m × 36 = 907.2 cum / hr.
5.i.3	Duration of Backwash	: 10 min/bed
5.i.4	Requirement per wash	: 907.2 / 60 × 10
		: 151.2 cum.
5.i.5	Provide capacity	: 151.2 × 2 = 302.4, Say 315 cum
5.i.6	Wash water tank filling pumps	: 160 cum/hr. (1W +1S) at 15 MWCX
	(To fill the tank in two hours)	
5.i.7	Backwash header to Filter Beds	: Dia 0.40 m (velocity = 2.00 m/sec)
5.i.8	WWT Head	: Effective head 8.50 m + pipe losses

	Frictional head loss in pipe	: Q = 907.2 cum/hr, Dia 0.40 m
		Hf = 14.2 m/1,000 m (From Tables)
5.i.9	Length of pipe	: 60 m
5.i.10	Total losses	: (14.2 / 1,000) × 60 = 0.85 m
5.i.11	Head of tank	: 8.50 + 0.85 = 9.35 m, say 9.50 m
5.j	**Air scour**	**:** Separate scour of both sections
5.j.1	Rate of air scour	: 36 cum/sq. m/ hr.
5.j.2	Capacity of blower	: 25.2 sqm × 36 = 907.2 cum/hr.
		: provide 910 cum / hr.
5.j.3	No of blowers	1 W + 1 S
5.j.4	Pressure	: 0.40 kg/sq. cm, (4.0 MWC)
5.j.5	Air scour header	: dia 150 mm
		(velocity = 14.26 m/sec)
5.k.	**Wash water troughs**	
5.k.1	No of troughs / bed	: 6, (six), three per section
5.k.2	Flow rate / trough	: 907.2 / 3 = 302.4 cum / hr.
		: 0.084 cum / sec
5.k.3	Size of trough	: $Q = 1.376\ b\ h^{3/2}$
	Q = 0.084 cum/sec	: $0.084 = 1.376 \times 0.35 \times h^{3/2}$
	b = width 0.35 m	: h = 0.31 m
5.k.4	Provide height of trough	: 0.35 m
5.k.5	Size of trough provided	: 0.35 m (width), 0.35 m (height)
5.k.6	Lateral travel of water	: {6.0 – (0.5 × 3)} / 6 = 0.75 m, hence OK
	(Length of filter bed = 6.0, Width of cross-trough = 0.50 m including wall thickness)	
5.l	**Central wash disposal gutter**	
5.l.1	Flow rate	: 907.2 cum/hr
5.l.2	Size of gutter	: $Q = 1.376\ b\ h^{3/2}$
	Q = 0.25 cum/sec	: $0.25 = 1.376 \times 0.60 \times h^{3/2}$
	B = width 0.60 m	: h = 0.45 m
5.l.3	Dia of disposal pipe	: 0.40 m (Velocity: 2.00 m/sec)
5.l.4	Velocity head ($h = V^2 / 2g$)	: $(2.60)^2 / (2 \times 9.81) = 0.20$ m
5.l.5	Total water depth in gutter	: 5.l.2 + 5.l.3 + 5.l.4
		(0.45 + 0.40 + 0.20 = 1.05 m)
5.l.6	Size of gutter provided	: width 0.60 m (ID), 1.00 m (OD)

		: height 2.10 m
5.m.	**Filter ports:**	
5.m.1	Filter inlet gate	: One per bed, Size 0.35 m × 0.35 m
	(Q = 230 cum /hr)	: (velocity = 0.52 m/sec)
5.m.2	Filter outlet, B/F valve	: Two per bed, dia – 0.20 m (200 mm)
	(Q = 115 cum/hr)	: (velocity = 1.02 m/sec)
5.m.3	Wash water inlet, B/F valve	: Two per bed, dia – 0.40 m (400 mm)
	(Q = 907.2 cum/hr)	: (velocity = 2.00 m/sec)
5.m.4	Wash water disposal, B/F valve	: One per bed, dia – 0.40 m (400 mm)
	(Q = 907.2 cum/hr)	: (velocity = 2.00 m/sec
5.m.5	Air scour inlet, B/F valve	: Two per bed, dia – 0.15 m (150 mm)
	(Q = 907.2 cum/hr)	: (velocity = 14.55 m/sec)
5.n	**Filter Underdrain system:**	for each section
5.n.1	Area of perforations	: 0.30% of bed (section) area
		: (25.2) × 0.30 / 100 = 0.076 sqm
5.n.2	Dia of perforations	: 10 mm each
5.n.3	No of perforations	: 962
5.n.4	Spacing of laterals	: 300 mm c/c on both sides
5.n.5	No of laterals	: 6.0 / 0.30 × 2 = 40
5.n.6	No of perforations / lateral	: 962 / 40 = 24
5.n.7	Length of lateral	: 1.95 m each
5.n.8	Spacing of perforations	: 1.95 / 24 × 2 = 0.162 m c/c in two rows
		Say 0.15 m c/c
5.n.9	Dia of lateral	: 90 mm OD Rigid PVC
		: 10 kg / sq. cm, (85 mm ID)
5.n.10	Total area of laterals	: 3.14 / 4 × (0.085^2) × 40 = 0.23 sqm
5.n.11	Ratio-area of laterals to area of perforations	: 0.23 / 0.076 = 2.98
5.n.12	area of manifold pit: 2 × area of laterals	: 2 × 0.23 = 0.46 sq. m
5.n.13	Provide Area of manifold pit	: 0.70 m × 0.70 m = 0.49 sq.
5.n.14	Ratio of area of manifold pit to area of laterals	: 0.49 / 0.23 = 2.13
5.o	**Filter inlet channel**	
5.o.1	No of channels	: 1, One
5.o.2	Flow rate / channel	: 920 cum/hr
5.o.3	Width	: 0.75 m
5.o.4	Velocity	: 0.60 m/sec

5.o.5	SWD	: 0.57 m, provide 0.65 m, FB 0.50 m
5.p	**Filter Inlet and Outlet weir (No of chambers = 4)**	
5.p.1	Flow rate	: 920 / 4 = 230 cum/hr
5.p.2	Head over weir	: Q = 1.84 b h $^{3}/^{2}$
	Q = 0.064 cum/sec	: 0.064 = 1.84 × 1.00 × h $^{3}/^{2}$
	B = length of weir 1.00 m	: h = 0.10 m
5.q	**Pure (Filtered) water channel**	
5.q.1	Flow rate	: 920 cum/hr
5.q.2	Width	: 1.25 m
5.q.3	Velocity	: 0.60 m/sec
5.q.4	SWD	: 0.34 m, say 0.45 m, FB 0.30 m
5.s	**Filtered water pipe to Pure Water Sump**	
5.s.1	No of pipes	: 1, One, Dia 0.60 m (600 mm)
5.s.2	Flow rate / pipe	: 920 cum/hr
5.s.3	Velocity	: 0.92 m/sec
5.t	**Pure Water Sump**	
5.t.1	Capacity of sump	920 cum (Detention time 1 hr.)
5.t.2	Side Water Depth	: 3.50 m
5.t.3	Diameter	: 18.3 m, say 18.50 m,
		FB 0.50 m
6	**Chemical / coagulant dosing system**	
6.a	**Alum dosing system**	
6.a.1	Dose considered	: 50 mg/Lit
6.a.2	Flow	: 920 cum/hr
6.a.3	Alum consumption	: 46 kg/hr
6.a.4	Strength of solution	: 10%
6.a.5	Rating of tanks	: 8 hr (3 no tanks)
6.a.6	Volume of alum solution per shift	: 46 kg/hr × 8 × (100 / 10)
		: 3,680 lit, say 4,000 lit
6.a.7	Dosing pumps	: 4,000/ 8 = 500 lit , say 0–750 lit/hr.
		(1W +1S) at 15 MWC
6.b	**Lime dosing system**	
6.b.1	Dose considered	: 25 mg/Lit
6.b.2	Flow	: 920 cum/hr
6.b.3	Lime consumption	: 23 kg/hr.
6.b.4	Strength of solution	: 10%

6.b.5	Rating of tanks	: 12 hr (2 no tanks)
6.b.6	Volume of lime solution per shift	: 23 kg/hr × 12 × (100 / 10)
		: 2,760 lit, say 2800 lit
6.b.7	Dosing pumps	: 2,800 / 12 = 233 lit, say 0–400 lit/hr.
		(1W +1S) at 15 MWC
6.c	**TCL Powder dosing system (Emergency Disinfection)**	
6.c.1	Dose considered	: 3 mg/Lit of free chlorine
6.c.2	Flow	: 920 cum/hr
6.c.3	TCL consumption	: 2.76 kg/hr.
6.c.3.a	Available chlorine	: 25%
6.c.3.b	Powder requirement	: 11 kg/hr.
6.c.4	Strength of solution	: 10%
6.c.5	Rating of tanks	: 8 hr (3 no tanks)
6.c.6	Volume of lime solution per shift	: 11 kg/hr × 8 × (100 / 10)
		: 880 lit, say 900 lit
7	**Pre- and Post-Chlorination**	
7.b.1	Pre - Dose considered	: 5 mg/Lit of chlorine
7.b.2	Flow	: 920 cum/hr
7.b.3	Chlorine requirement	: 4.6 kg/hr., say 5 kg/hr.
7.b.4	No of chlorinators	2, Two (1W + 1S)
7.b.5	Post - Dose considered	: 3 mg/Lit of chlorine
7.b.6	Flow	: 920 cum/hr
7.b.7	Chlorine requirement	: 2.76 kg/hr, say 3 kg/hr.
7.b.8	No of chlorinators	2, Two (1W + 1S)
8	**Wash water re-circulation sump**	
8.a	Capacity required	: 315 cum (For one filter backwash)
		Provide capacity: 325 cum
8.b	SWD	: 2.50 m
8.c	Dia	: 13.0 m, FB : As per site condition
8.d.	**Wash water re-circulation pumps**	
8.d.1	Quantity of one filter backwash (10 min)	: 315 cum
8.d.2	No of filters backwashed per day	: 315 × 4 beds in one day = 1,300 cum
	(Worst case assumed for design)	
8.d.3	Re-circulation operation period	: 16 hr
8.d.4	Re-circulation flow rate	: 1300 / 16 = 81.25 cum/hr
8.d.5	Capacity of pumps provided	: 85 cum/hr
8.d.6	No of pumps	: 2, Two (1W + 1S) at 20 MWC
9	**Sludge Treatment & Disposal**	

	Sludge Mass Balance: Design Capacity:	
A)	TSS in Raw water (Average)	: 100 mg/L (Silt and Turbidity)…..Assumption
	TSS due to Alum dose (50 mg/L)	: 13 mg/L (50 × 0.26)
	TSS due to lime dose (25 mg/L)	: 25 mg/L
	Total TSS	: 138 mg/L, say 150 mg/L
	Rate plant capacity	: 920 cum/hr.
	Sludge consistency from clarifiers	: 2% W/V (20,000 mg/L)
	Mass balance:	
	Total Input solids = Total output solids + Total solids in the sludge	
	[(920) 150 = (920 – s) × 10] + 20000 s	
	Where s: sludge flow from clarifiers, cum/hr	
	Hence, s = 7.0 cum/hr	
B)	Thickener underflow consistency: 5% W/V (50,000 mg/L)	
	Mass balance:	
	(20,000 × 7.0) = (7.0 - s) 100 + 50000 s	
	Where s: sludge flow from Thickeners, cum/hr	
	Sludge flow to Thickener	= 3.0 cum/hr
	Hence, s (Feed to centrifuge)	= 3.0 cum/hr
	No of Centrifuges	= 1W +1S
	Overflow (supernatant) from Thickener	: 7.0 – 3.0= 4.0 cum/hr
C)	Centrifuge treated sludge consistency	0.25
	Centrate from Centrifuge: 3.0 × 0.75	= 2.25 cum/hr
10.	**Sludge Sump for clarifiers:**	
10.a	Flow Rate (sludge)	: 7.0 cum/hr
10.b	Detention time	: 2 hrs.
10.c	Capacity	: 14.0 cum
		(Size: Suitable)
10.d	Sludge pumps	: 7.0 cum/hr at 20 MWC,
		(1W +1S)
11	**Sludge Thickeners**	
11.a	No of units	: 1, One
11.b	Flow Rate (Sludge from clarifiers)	: 7.0 cum/hr
11.c	Solids loading	: (7.0 × 20,000 × 24) / 1000
		= 3,360 kg/day
11.d	Solids loading rate	: 80 kg/sqm/day
11.e	Area required	: 3,360 / 80
		= 42.0 sqm
11.f	Dia of unit	: 7.31 m, say 7.50 m, F.B. 0.50M, each

11.g	SWD	4.0 m
12	**Thickened sludge sump**	
12.a	Flow Rate (sludge)	: 3 cum/hr
12.b	Detention time	: 2 hrs.
12.c	Capacity	: 6.00 cum
		(Size: Suitable)
12.d	Sludge pumps	: 3.0 cum/hr at 20 MWC (16 hr.),
		(1W +1S)
13	**Centrifuge :**	
	Capacity	: 3.0 cum/hr (1W+1S)
14	**Polyelectrolyte for Centrifuge**:	
14.a	PE Dose	: 1 kg/tonne of dry solids
14.b	Dry solids per day	: 3.36 tonne
14.c	PE requirement per day	: 3.36 kg
14.d	Strength of solution	: 0.5%
14.e	Requirement of solution	: (3.36 × 100) / 0.5
		: 672 lit, say 700 lit
14.f	No of tanks	: 1, one
14.g	Capacity of pumps	: 0–50 lit/hr. (1W + 1S)
	Note :	
	1.Hydraulic flow diagram (calculations for losses) to be developed with 20% overloading,	
	i.e., 920 × 1.2 = 1,104 cum/hr.	
	2. The solved example is only for the guidance showing approach to the sizing calculations. The designer may improve on it with better engineering practices.	
	Abbreviations :	
	Cum/hr – cubic metre per hour	
	SWD – side water depth	
	FB – freeboard	
	Cuft/sec – cubic feet per sec	
	RPM – rotations per min	
	Sqm – square metre	
	Rm – running metre	
	Dia. – diameter	
	M.W.C. – Metre Water Column	
	F.S.L. – Full Supply Level	
	T.W.L. – Total water level	
	W – Working	
	S – Standby	

Annexure 8.2: Sizing for Tube-Clariflocculator

(Concentric flocculation zone with Peripheral annular tube settling zone)

a	No of units proposed	: 1, one
b	Capacity / Flow per unit	: 920 cum/hr
c	Diameter of central shaft	: 0.90 m (ID), 1.30 m (OD)
	(ID –Internal Dia, OD –Outer Dia.)	(1.30 = 0.90 + 0.20 + 0.20)
d	Velocity through shaft	: 0.40 m / sec
e	Size of outlet openings	: 0.60 m × 0.30 m × 4 no
f	Velocity through openings	: 0.35 m / sec
g	Let Diameter of flocculator be	: 13.00 m (ID),
h	Side water depth	: 3.50 m
i	Volumetric capacity	: 3.50 × 3.14/4 × (13.00^² - 1.30^²)
		: 461.3 cum
J	Detention time	: (461.3/920.) × 60 = 30.05 minute
K	Surface area of flocculation zone (A1)	: 461.3 / 3.5 = 131.8 sqm
l	Surface loading rate on Tube Settling	: Peripheral Annular zone
	clarification zone	: 5.0 cum/sqm/hr
	(Tube size 50 mm × 50 mm, Length 600 mm, inclination 60 degree)	
	i. Surface area of clarification (A2)	: 920 / 5.0 = 184 sqm
	ii. Total area of clariflocculator (A1 + A2)	: 131.8 + 184 = 315.80 sqm
	iii. Diameter of clariflocculator (ID)	: sqrt (315.80 × 4 / 3.14) = 20.04 m Say 20.10 m
m	Detention time in clarifier zone Actual volume in clarification zone	: swd 3.75 m :
		: $(20.10^2 - 13.0^2) \times (3.14/4) \times 3.75$
	Detention time	= 691.85 cum
		: 691.85 / 920
		= 0.75 hr (45 min)
n	No of radial collection troughs	20
o	Length of trough	= (20.10 – 13.0) / 2 = 3.55 m
p	1. Collection weir length/trough	= 3.55 m
	Equi-spaced 90-degree V-notches (50 mm depth) as collection edges	
	2. Total weir length	: 3.55 × 20 × 2 = 142 rm
q	Weir loading rate:	= (920 × 24) / 71 = 300 cum/rm/day
	i. No of slow mixers/flocculators	2, two per unit
	ii. G value of flocculator agitator	: 45–50 m/sec/m
r	Size of Outlet openings from flocculator to clarification zone: (At bottom)	
	i. Velocity through opening	: 0.005 m/sec
	ii. Area of opening	: (920/3,600) / 0.005
		= 51.11 sqm
	iii. Perimeter of flocculation wall	: 14.40 × 3.14 = 45.21 m
	iv. Height of opening	: 51.11 / 45.21 = 1.13 m, Say 1.15 m

	(Flocculator partition wall fabricated out of 5 mm FRP sheets, supported by MS frame. The partition wall is supported by horizontal cantilevered trusses fixed to the outer RCC wall, below the modules.)	
	Configuration of tube settling zone:	
	1. Depth of water from collection troughs to top of tube modules: 0.90 m	
	2. Height of tube modules: 0.52 m (Slant length 0.60 m)	
	3. Depth of water below modules [3.75 m – (0.90 m + 0.52 m)] = 2.33 m	
	4. Total depth = 0.90 + 0.52 + 2.33 = 3.75 m	
	5. Bottom slope 1:12 towards centre	
	6. Sludge scrapper bridge: Either centrally driven or peripherally driven	
s.	**Collection launder (Outside)**	: Two semi-circular channels
	i. Flow per channel	: 920 / 2
		: 460 cum/hr
	ii. Width	: 0.75 m
	iii. Velocity	: 0.60 m/sec
	iv. SWD (Side Water Depth)	: 460.4 / (3600 × 0.75 × 0.60)
		: 0.28 m, say 0.40 m
	Note :	
	1. Sometimes, tube module area is provided 10% more than the actual requirement to compensate for dead portion of the fabricated modules.	
	2. Flocculation tanks and tube settling tanks can also be configured in a rectangular shape with a common partition wall. In such a case, multiple hoppers are provided at the bottom of tube settlers with slope not less than 50 degrees for sludge removal.	

Annexure 8.3: Information to be Included in WTP Tender

GENERAL

The principal requirement must be a spacious and convenient layout. The structures should represent a pleasing appearance with aesthetic features forming a balance between function and form. The interiors of the structures shall be eye appealing and in keeping with the objectives of the plant, viz., production of pure and wholesome water.

While the mode of design and construction could be a matter of individual choice, it should be ensured that all materials, design, construction, and fabrication details for different units including doors and windows conform to the relevant IS specifications and codes of practice wherever available and in their absence, to the established standards.
Adequate provision shall be made in the civil engineering works for laboratory, office buildings, administration area, sanitary facilities, water supply, etc. The area requirement of these ancillary requirements shall be stipulated. Roadways with adequate lighting shall be provided. Adequate ladders or steps and handrails, where required, shall be provided for easy access to each unit of the treatment plant and wherever necessary, walkways should be provided. Interconnecting facilities shall be provided to enable the operator to move freely for maintenance and operation of the plant.

All water retaining structures shall be designed in conformity with IS: 3370 (Part 1) (2009); IS: 3370 (Part2) (2009); IS: 3370 (Part 3) (1967, reaffirmed 2008); IS 3370 (Part 4) (1967, reaffirmed 2008) while the other structures shall be designed according to IS 456 (2000, reaffirmed 2005).

The tender specifications should include, inter alia*,* process requirements and specifications for equipment.

A. Process Requirements

1. The following data shall be furnished to the renderers:

   a) Raw water analysis comprising of monthly average figures preferably for a full year period covering various seasonal variations in respect of, at least the following. If the full year data is not available, the worst seasonal values may be given:

      i. pH
      ii. Turbidity
      iii. Total Alkalinity
      iv. Total hardness
      v. Chlorides
      vi. Coliform organism (MPN)

   b) Any other additional data, if the water is known to contain constituents or contaminants which are required to be removed:

      i. Phenols
      ii. Tastes and odours
      iii. Colour
      iv. Carbon dioxide
      v. Algal content
      vi. Iron

vii. Manganese
viii. Hardness (Carbonate and non-carbonate along with magnesium content of water)
ix. Fluoride content
x. Chlorine demand and any other pollutants arising from industrial effluents and agricultural runoff

c) Hydraulic data such as the relevant raw water inlet and filtrate outlet levels.

Head loss in treatment plant, i.e., difference between lip of aeration fountain and FSL of pure water sump is a vital parameter, as saving in it reduces head on pumping machinery of raw water and pumping water put together. As this saving is for total quantity to be treated and as the amount saved is on recurring basis, saving in electrical energy is sizably high for every meter of reduction in loss of head in treatment plant. Hence allowable total head lass in WTP should be judiciously worked out by ULB before tendering. As a thumb rule, total head loss from lip of aeration fountain to FSL of pure water sump should not be more than 5 m. In case of diffused aerator /stilling chamber the figure of 5 m will be on lower side. Even if land for construction of WTP is having more slope the pumping energy should not be wasted and providing of proper layout should be insisted by providing proper FSLs of various units of treatment

d) The following requirements shall be furnished:

i. The flow requirements of the plant in terms of the net output expected of the plant for a given period of time, say 23.5 hours a day (allowing for washing of the filters, etc., and also overload capacity.)
ii. The quality of the treated water in terms of pH, turbidity, coliform organisms (MPN) and E. coli, and, where needed, iron, manganese, hardness (carbonate and non-carbonate), magnesium content, fluoride content, and colour.
iii. Design parameters for various treatment units such as coagulants and coagulation aids dosing, rapid mixing, slow mixing, sedimentation, filtration, and chlorination, as well as special processes like aeration, micro-straining, iron and manganese removal, fluoride removal, taste and odour control as per specific local requirements and in accordance with the details furnished in the manual.
iv. A suggested, layout of a water treatment plant including the following details, to the extent possible:

1. unit sizes and location of plant structures;
2. schematic flow diagram showing flow through various units;
3. piping arrangement including bypasses showing the material and size of pipes as well as direction of flow;
4. hydraulic profile of the units showing the flow of water;
5. contour map of the area including provision for future expansion;
6. approach roads and water supply facilities for construction purposes;
7. other information about site such as proneness to flooding and earthquakes, groundwater table fluctuations, type and nature of soils met up to maximum anticipated depths, soil characteristics like bearing capacity and corrosivity, intensity and duration of rainfall and total annual rainfall, locations of areas for disposal of excavated spoils and of borrow pits, if required, for filling purposes.

The contract should establish where guarantees apply and clearly define their requirements. Performance guarantees must be demonstrated by a test run of specified length or over an agreed period of operation.

B. Mechanical Equipment

The following data may be given while inviting tenders for pumping plant

a) Number of units required to work in parallel.

b) Nature of liquid to be pumped:

   i. fresh or salt water;
   ii. temperature of liquid;
   iii. specific gravity;
   iv. amount of suspended matter present.

c) Required capacity as well as minimum and maximum amount of liquid the pump must deliver.

d) Suction conditions:

   i. suction lift or suction head;
   ii. constant or variable suction condition.

e) Discharge conditions:

   i. maximum/minimum discharge pressures against which pump has to deliver liquid;
   ii. static head description: constant or variable;
   iii. friction head description and how estimated.

f) Type of service: continuous or intermittent.

g) Pump installation: horizontal or vertical position (if vertical type of pit, wet and dry).

h) Power available to drive the pump.

i) Space, weight, or transportation limitations.

j) Location of installation.

k) Special requirements with respect to pump design: construction or performance. The following requirements may be indicated.

   i. The pump equipment as well as the component shall conform to the relevant IS standards and in their absence, to any other accepted international or national standard.
   ii. Any special duty conditions such as temperature, humidity, corrosive atmosphere should be specified.
   iii. Submerged structure parts except hot rolled sections shall not be less than 6 mm thick under normal atmosphere and 8 mm in aggressive atmospheres.
   iv. Prime movers and allied components such as electrical motor, starter switches reduction gear, drive mechanism, bearings, and plummer blocks shall be of approved make.
   v. All rotating machinery particularly gears shall be designed with adequate safety margins and service factors.

      a) An item-wise pricelist of spare pans shall be furnished by the tenderer. At least two years requirement of fast-moving spares should be supplied along with the equipment.

b) The supplier of special equipment like softeners, recording gauges, rate controllers, chlorinators, proportioning coagulant feeders, meters, etc. shall furnish the services of a competent representative for a specified number of days during a specified period to instruct the plant operating personnel in the maintenance and care of the equipment and to conduct tests and make recommendations for producing most efficient results.

c) Equipment selection with respect to specification, spare units, spare parts, and servicing can affect maintenance, operating and investment costs. It is the purchaser's responsibility to incorporate into the contract all requirements and limitations which affect cost. Equipment performance is usually guaranteed by the manufacturer.
The contractor shall furnish bonds converting items of work like mechanical equipment, piping, etc., for specified period as a guarantee of satisfactory operation and correction of any defect in the work, material, or equipment furnished by them.
On special equipment extended guarantees, maintenance over a period of time and supervision of a complete installation may be provided by the manufacturer. On most large equipment, the manufacturer provides field service with respect to installation.

d) All submerged water parts, rotating mechanical parts, and steel pipes shall be adequately protected after surface preparation. Oil, grease, dirt, soil, and all surface contaminants from structural and fabricated steel parts are removed by cleaning with solvent vapour, alkali emulsion, or steam. Loose rust or paint, weld spatter, etc., are removed by hand chipping, scraping, sanding, wire brushing and grinding. The bare finished shaft, finished flanges, and other mechanical surfaces are protected by grease line or rust protection measures. Structural mechanism support and super structure, walkway, handrails, fabricated shafts, etc., shall be protected with at least one coat of primer and two coats of paint.

e) The below listed IS codes are recommended for resilient building structures in water treatment plants, water-retaining process units, along with other annexing units:
   i. IS 456: 2000 (Reaffirmed Year: 2021) - Plain and Reinforced Concrete - Code of Practice (Including Amendment 1, 2, 3, & 4);
   ii. SP 20 (1991): Handbook on Masonry Design and Construction;
   iii. SP 23 (1982): Handbook on Concrete Mixes;
   iv. IS 800: 2007 (Reaffirmed Year: 2017) - General Construction in Steel - Code of Practice gives the code of practice for use of structural steel in general building construction;
   v. IS 875: Part 1: 1987 (Reaffirmed Year: 2018) - Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures Part 1 Dead Loads - Unit Weights of Building Material and Stored Materials (Incorporating IS 1911: 1967)-refers to dead loads other than earth quake;
   vi. IS 875: Part 2: 1987 (Reaffirmed Year: 2018) - Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures: Part 2 Imposed Loads-refers to imposed loads;
   vii. IS 875: Part 3: 2015 (Reaffirmed Year: 2020) - Design Loads (Other than Earthquake) for Buildings and Structures - Code of Practice Part 3 Wind Loads (Third Revision)-refers to wind loads;

viii. IS 875: Part 4: 2021 - Code of Practice for Design Loads other than earthquake for Buildings and Structures Part 4 Snow Loads - refers to snow loads;
ix. IS 875: Part 5: 1987 (Reaffirmed Year: 2018) - Code of Practice for Design Loads (Other Than Earthquake) For Buildings and Structures Part 5 Special Loads and Combinations
x. IS 4326: 2013 (Reaffirmed Year: 2018) - Earthquake resistant design and construction of buildings - Code of practice
xi. IS 1893: 1984 (Reaffirmed Year: 2022) - Criteria for Earthquake Resistant Design of Structures (Fourth Revision)
xii. IS 1893: Part 1: 2016 (Reaffirmed Year: 2021) - Criteria for Earthquake Resistant Design of Structures - Part 1: General Provisions and Buildings
xiii. IS 1893 Part 2: 2014 (Reaffirmed Year: 2019) - Criteria for Earthquake Resistant Design of Structures Part 2 Liquid Retaining Tanks
xiv. IS 1893: Part 3: 2014 (Reaffirmed Year: 2019) - Criteria for Earthquake Resistant Design of Structures Part 3 Bridges and Retaining Walls
xv. IS 1893: Part 4: 2015 (Reaffirmed Year: 2020) - Criteria for Earthquake Resistant Design of Structures Part 4 Industrial Structures Including Stack - Like Structures (First Revision)
xvi. IS 1893: Part 6: 2022 - Guidelines for Criteria for Earthquake Resistant Design of Structures Part 6: Base Isolated Buildings

f) In addition, below listed IS codes are recommended to be followed for reinforcement detailing and guidance:
i. SP:34 (S&T) – 1987 - Handbook on concrete reinforcement detailing.
ii. IS 432: Part 1: 1982 (Reaffirmed Year: 2020) - Mild Steel and Medium Tensile Steel Bars and Hard-Drawn Steel Wire for Concrete Reinforcement: Part 1 Mild Steel and Medium Tensile Steel Bars
iii. IS 432: Part 2: 1982 (Reaffirmed Year: 2020) - Mild Steel and Medium Tensile Steel Bars and Hard-Drawn Steel Wire for Concrete Reinforcement: Part 2 Hard-Drawn Steel Wire
iv. IS:1786-2008: (Reaffirmed Year: 2018) - Specification for High strength deformed steel bars and wires for concrete reinforcement
v. IS: 2502-1963(Reaffirmed Year: 2018) - Code of practice for bending & fixing of bars for concrete reinforcement.
vi. IS: 2751 -1998(Reaffirmed Year: 2019) - Recommended practice for welding of mild steel plain & deformed bars for reinforced construction.
vii. IS: 5525 -1969(Reaffirmed Year: 2018) - Recommendation for detailing of reinforcement in reinforced concrete works.
viii. IS: 9077 -1979 (Reaffirmed Year: 2018) - Code of practice for corrosion protection of steel reinforcement in RB & RCC construction.

Annexure 8.4: Common Chemicals in Water Treatment

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
1	Activated Carbon		C	Taste and Odour control, de-chlorination	Granular	Not less than 80% C	Black granules 1–3 mm	Water passed through granular beds	Dry		
2	Activated Carbon		C	Do	Black powder	Do	200 to 400 μm black powder insoluble		Dry or in slurry form careful mixing required to maintain proper slurry	Iron or Steel Tank	
3	Activated Silica	Silica Sol.	SiO_2	Coagulant aid	Produced at site as needed from sodium silicate		Clear, often opalescent	0.6	Wet batch made up by pH adjustment and aged	Mild steel or stainless steel or rubber contain	Appliances likely to be clogged with improper pH

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
					and activating agents					ers	adjustment or feed
4	Aluminium Sulphate	Alum; filter alum; sulphate of alumina	$Al_2(SO_4)_3.4H_2O$	Coagulant	Blocks; Lumps; Powder	At least 16% Al_2O_2	Light tan to grey; crystalline; acidic; corrosive; slightly hygroscopic	8%–10%	Wet or Dry	Acid proof brick tanks, bitumen coated concrete or rubber lined tanks	
5	Aluminium Sulphate (ferric)	Alum; filter alum; sulphate of alumina; alumina ferric	$Al_2(SO_4)_3.4H_2O$ (approx.)	Do	Do	15% (approx.) Al_2O_2	Brown to dark brown; crystalline; acidic; corrosive; hygroscopic	8%–10%	Do	Do	High concentration of Iron

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
6	Filter Alum	Filter Alum	$Al_2(SO_4)_3.18H_2O$	Coagulant	Solution Sp. G. 1.1	8% Al_2O_2	Brown solution; acidic; corrosive	Direct or in 1% solution	Wet, orifice box rotameter and proportionating pumps	Acid proof brick tanks, bitumen coated concrete or rubber lined tanks	Costs less than dry alum if close enough to source of manufacture
7	Ammonium Sulphate	Sulphate of ammonia	$(NH_4)_2SO_4$	Chloramine treatment in disinfection	Crystal	20%–25% NH_3	White sugar sized crystals	0.1 to 0.5%	Wet proportionating pumps	Stainless or plastic containers	
8	Anhydrous ammonia	Ammonia	NH_3	Do	Liquified Gas	98%–99% NH_3	Colourless gas; pungent irritating offensive		Wet ammoniator	Iron, Steel, or glass	Dangerous coagulant

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
							odour				
9	Bentonite	Colloidal clay	H_2O (Al_2O_2, Fe_2O_3, MgO) $4SiO_3.nH_2O$	Coagulant aid, floc weighting agent	Powder pellets		Yellow Brown		Wet suspension	Iron or steel	
10	Calcium Hydroxide	Hydrated lime slaked	$Ca(OH)_2$	pH adjustment and softening	Powder	80%–90% $Ca(OH)_2$	White powder; caustic	1.50%	Dry or wet. Can be fed in suspension	Iron steel or concrete tanks	Not very soluble, lead tank cannot be used
11	Calcium Hypochlorite	HTH, per chlorine	$Ca(OCl)_{2.}4H_2O$	Disinfectant taste or odour control	Granular powder	70% available chlorine	White	2 to 4%	Wet	Stoneware, plastic, rubber tank	Dangerous coagulant

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
12	Calcium Oxide	Quick lime, burnt unslaked lime	CaO	pH adjustment and softening	Pebble crushed lumps to powder	40%–90% CaO	White or light grey, caustic	1.50%	Dry or wet. Can be fed in suspension. Dry feeders generally discharge to slake before application	Iron, steel, or concrete tanks	Lead tank cannot be used
13	Chlorine	Chlorine gas, Liquid chlorine	Cl_2	Disinfectant taste and odour control, general oxidant	Liquified Gas under pressure	99%–99.8% Cl_2	green yellowish gas, pungent, corrosive, heavier than air, health hazard		Wet chlorinator	Dry iron copper steel solution, silver glass, hard rubber, lead special alloys	Dangerous coagulant very careful handling required. Should use gas mask

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
14	Chlorinated ferrous sulphate	Chlorinated copperas	$Fe_2(SO_4)_3$ $FeCl_2$	Coagulant	Yellow solution	Produced at site by reaction of chlorine and ferrous sulphate		3%–5%	Wet	Rubber lined or stainless steel containers, plastic containers	
15	Chlorinated lime	Bleaching powder, chloride of lime	Cao, $2CaOCl_2.H_2O$	Disinfection	Powder	25%–33% available chlorine	White, hygroscopic, unstable pungent powder	1%–2%	Wet	Plastic, stoneware, or rubber tanks	
16	Chlorine dioxide		ClO_2	Taste and odour control disinfection	Gas	26.3% available chlorine	Generated at site	0.10%	Wet	Plastic, soft rubber	

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
17	Copper sulphate	Blue vitriol	$CuSO_4.5H_2O$	Algicide	Crystal lumps, powder	90%–95%	Clear blue crystals	1%–2%	Dry put in bags, dragged behind boat, dusted on surface with special equipments	Stainless steel, plastics	
18	Ferric Chloride	Chloride of Ion	$FeCl_2.6H_2O$	Coagulant	Lump sticks crystal	60% $FeCl_3$	Yellow Brown highly hygroscopic, very corrosive	3%–5%	Wet Proportionating pump	Rubber lined tank or stoneware containers plastic	-
19	Ferric Chloride (Solution)	Do	$FeCl_2$	do	Solution	30%–40% $FeCl_3$	Brown Solution, Very Corrosive	3%–5%	Wet Proportionating pump	do	-

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
20	Ferric Sulphate	Iron Sulphate, Ferrifloc	$Fe_2(SO_4)_3.H_2O$	Coagulant	Granules crystal lumps	18.5–0.1% Fe	Red brown or grey crystals mildly hygroscopic	3%–5%	wet	Dry; iron stainless steel and concrete wet: lead or stainless steel plastic	Solution is Corrosive
21	Ferrous Sulphate	Coappears green vitriol; sugar Sulphate	$FeSO_4.7H_2O$	do	do	20% Fe	Green to Brownish yellow crystals hygroscopic	+8%	wet	do	Cakes in storage above 20° lime addition necessary
22	Hydrazine hydrate	-	$H_2H_4H_2O$	Deoxygenation	Solid Powder	64%	White Powder	-	Wet Proportionating pump	Stainless steel Pump	plastics
23	Hydrochloric Acid	Muriatic acid	HCl	in Cation exchange	Regenerant	Liquid	30% HCl liquid	5%–10%	Wet pump	Glass rubber lined	Dangerous Coagulant

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
24	Potassium Aluminium Sulphate	Potash Alum	$K_2SO_4.\ Al_2(SO_4)_3.\ 24H_2O$	Coagulant	Lump powder, blocks	8%–11% Al_2O_3	White crystalline, acidic corrosive, hygroscopic	1%	Wet or dry		Generally used in business filters
25	Sodium Aluminate	Soda Alum	$Na_2Al_2O_3$	Coagulant	Crystalline flakes, solution	43%–45% Al_2O_3	White to grey crystal, liquid caustic, alkaline	5%	Wet or dry	Cast iron mild steel concrete	Generally used along with filter alum
26	Sodium Carbonate	Soda Ash	Na_2CO_3	pH adjustment softening	Dense crystal, Powder	98%–99% Na_2CO_3	White powder, Caustic	1%–10%	Dry or wet Proportioning pump		Dangerous Coagulant
27	Sodium Chloride	Common Salt	NaCl	Softening regenerant	Crystals	90%–95% NaCl	Colourless crystals	8%-10%	Wet, fed through injection nozzles	Bitumen or epoxy coated M.S. tanks	

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
28	Sodium Chromate	Chromate	$NaCrO_4$	Corrosion preventive anodic inhibition	Crystals	80%–90% Na_2CrO_4	Yellow/Brown Crystals	-	Wet Proportionating pump	Stainless steel, plastic	Dangerous Coagulant, harmful to eyes. Used in combination with sodium silicate
29	Sodium Hexa-meta phosphate	Calgan, glassy phosphate	$(NaPO_2)_4$	Scale and corrosion prevention	Powder flakes	60%–63% P_3O_2	Like broken glass	0.25	Wet Proportionating pump	Stainless steel, plastic, hard rubber	Holds up ppm of Fe, Mn, Ca, and Mg, for M.S. protective coating
30	Sodium Hydroxide	Caustic Soda	NaOH	pH adjustment, softening and filter cleaning	Flakes, lumps, pellets, powder	96%–99%	Alkaline, corrosive, hygroscopic	1%–10%	Wet Proportionating pumps, orifices box, rotameter	Cast iron, mild steel, rubber lined	Dangerous Coagulant

S. No.	Chemical	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
31	Sodium Hydroxide (Solution)	Caustic Soda, lye	$NaOH$	Do	Solution	10%–40% $NaOH$	Syrupy solution	Do	Do	Do	Do
32	Sodium Sulphate		Na_2SO_4	Deoxygenation	Powder, lumps	90%–99% Na_4SO_2	White powder	1%	Wet Proportionating pump	Stainless steel, plastic	8 mg/L sodium sulphate required to remove 1 mg/L O_2
33	Sulphur dioxide		SO_2	De-chlorination or filter cleaning	Gas	100%	Colourless, pungent	-	Dry	Steel	
34	Sulphuric Acid	Vitriol	H_2SO_4	pH adjustment, lowering of alkalinity	Liquid	60%–90% (commercial H_2SO_4)	Syrupy, corrosive, hygroscopic	1%–2%	Wet, dilute solution, orifice box, rotameter	Lead, porcelain, or rubber	Always add acid to water, dangerous coagulant
35	Sulphuric Acid	Do	Do	regenerant Cation exchange operation	Do	98% H_2SO_4	Do	Do	Wet pumping	Lead, glass, or rubber lined tank	Do

Annexure 8.5: Recommended Design Criteria for WTP

S. No.	Description	Range of values
Unit Process: Cascade Aerator		
1.	Area loading rate	0.015–0.045 sqm/cum/hr
2.	No of steps	4–6
3.	Rise of steps	0.15 m to 0.25 m
4.	Tread of steps	Minimum, twice the rise
5.	Total drop over cascades	0.60 m to 1.50 m
6.	Velocity in central shaft	Less than 0.60 m/sec
7.	Shape of the unit	Circular
Unit Process: Coagulation		
A. Mechanical Flash Mixer		
1.	Detention time	60 sec
2.	Velocity Gradient	300 m/sec/m
3.	Paddle tip velocity	3 to 4 m/sec
4.	Ration of Depth to Dia	1:1 to 3:1
5.	Shape	Circular or Square
B. Hydraulic Weir Mixer		
1.	Drop over weir (freefall)	0.4 to 0.60 m
2.	Velocity gradient	800–1,000 m/sec/m
3.	Detention time in downstream chamber	2–5 sec
4.	Detention time in upstream chamber/channel	Minimum 1.0 min
Unit Process: Flocculation		
Mechanical Flocculator		
1.	Detention time (t)	20–40 min
2.	Velocity Gradient (G)	40–70 m/sec/m (Variable speed)
3.	Product ‘Gt’	2 to 6 × 10^4
4.	Paddle tip velocity	0.40 to 0.70 m/sec
5.	Area of paddles	10%–25% of cross-sectional area of the tank
6.	Shape	Circular, Square, or Rectangular
Unit Process: Clarification or sedimentation		
A. Clariflocculators		
1	Surface loading rate	30–40 cum/sqm/day
2.	Side Water Depth	Minimum 3.50 m
3.	Detention time	2–3 hrs
4.	Weir loading rate	200–300 cum/RM/day
5.	Inlet velocity (flocculated water)	Maximum 0.005 m/sec
6.	Sludge storage depth	Volume of Conical bottom below SWD

7.	Velocity of water in central shaft	Max 0.60 m/sec
8.	Velocity of water at inlet ports	Max 0.40 m/sec
9.	Velocity in Inlet Pipe to Clariflocculator	0.80 m/sec to 1.20 m/sec
Tube Settling Tanks Tube size 50 mm × 50 mm or equivalent shape, Tube length 600 mm, Angle of inclination 60 degree		
1.	Surface loading rate	5–6 cum/sqm/hr (5,000–6,000 lph/sqm)
2.	SWD	3–4 m (above hoppers or scrapper)
3.	Detention time	30–45 min
4.	Weir loading rate	300–400 cum/RM/day
Unit Process: Granular Media Gravity Filtration		
Rapid Sand Gravity Filter beds		
1.	Rate of filtration	5–6 cum/sqm/hr (5000 to 6000 lph/sqm)
2.	Ratio of Length to width of Filter box (Two sections with central gullet)	1 to 1.6
3.	Depth of water over media	2 to 2.5 m
4.	Depth of sand	0.60 to 0.80 m (E.S. 0.55 to 0.6 mm), U.C. (1.5 to 1.7)
5.	Depth of supporting gravel	0.45 to 0.60 m, Graded
6.	Rate of backwash (water wash)	35–45 cum/sqm/min
7.	Head of backwash water tank	Effective head 8.50 m above filter bottom + losses in piping
8.	Duration of backwash	8 to 10 min
9.	Rate of air scour	35–45 cum/sqm/min
10.	Pressure rating	0.35 to 0.40 kg/sqcm
11.	Filter underdrain	Manifold block and lateral pipes
12.	Lateral travel of wash water	0.60 to 1.0 m
13.	Distance between top of the media and crest of the trough	0.65 m to 0.75 m
14.	Filter Control System	Constant Rate Filtration with Influent splitting weirs
15.	Height of the filter outlet weir above media top	0.10 to 0.15 m
16.	Filter Port Velocities	
	Inlet	Up to 1.0 m/sec
	Outlet	Up to 1.0 m/sec
	Wash Inlet	Up to 3.0 m/sec
	Wash Disposal	Up to 2.5 m/sec
	Air scour piping	Up to 25 m/sec
Unit Process: Disinfection		
Chlorine Contact Tank		
1.	Shape	Square or Circular

2.	Detention time	30 min
3.	No of baffles	4 to 6
Note: For Additional details, please refer to the body text of Manual		

Annexure 10.1: Chemical Dosages in Water Softening

1. Problem Statement

The water with the following chemical constituents is to be softened using the lime-soda process. Compute the quantities of chemicals required to treat 250 m^3/h of water flow, assuming practical units of removal for $CaCO_3$ to be 30 mg/L and for $Mg(OH)_2$ to be 10 mg/L.

CO_2	= 10.9 mg/L	Alkalinity (CO_2)	= 175 mg/L
Ca^{++}	= 100 mg/L	SO_4^{2-}	= 107 mg/L
Mg^{++}	= 8.9 mg/L	Cl^-	= 17.8 mg/L
Na^+	= 11.5 mg/L		

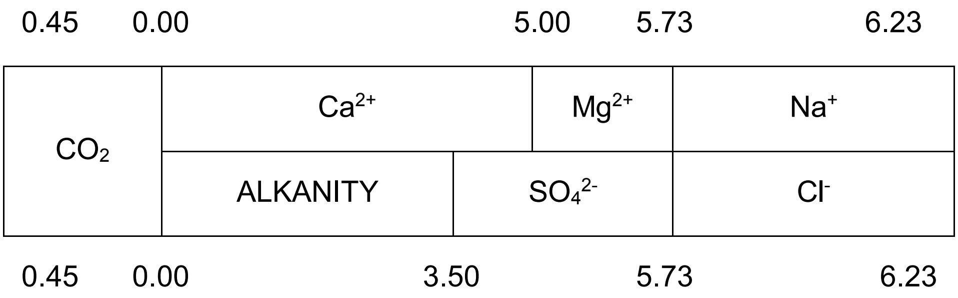
2. Solution

a) Compute mg/L of all components present in water.

S. No.	Component	Concentration, mg/L	Equivalent weight	Conc. mg/L
1	CO_2	10.9	22.0	0.45
2	Ca^{+2}	100.0	20.0	5.00
3	Mg^{+2}	8.9	12.3	0.73
4	Na^+	1.5	23.0	0.50
5	Alkalinity	175.0	50.0	3.50
6	SO_4^{2-}	107.0	48.0	2.23
7	Cl^-	17.8	35.5	0.50

b) Prepare an mg/L bar graph of raw water with a hypothetical combination.

In preparing such a bar graph, the concentration of cations is usually arranged left to right starting with Ca^{2+} followed by Mg^{2+}, Na^+, etc., in that order. Similarly, below the cations, anions are arranged left to right commencing with alkalinity followed by SO_4^{2-}, Cl^-, etc. The CO_2 being a molecule is conventionally shown to the left of the zero mark.



Graph

3. Compute the quantities of chemicals required.

a) Chemical reactions:

CO_2	+	$Ca(OH)_2$	$\rightleftharpoons$	$CaCO_3 + H_2O$	(1)
$Ca(HCO_3)_2$	+	$Ca(OH)_2$	$\rightleftharpoons$	$2CaCO_3 + 2H_2O$	(2)
$MgSO_4$	+	$Ca(OH)_2$	$\rightleftharpoons$	$Mg(OH)_2 + CaSO_4$	(3)
$CaSO_4$	+	Na_2CO_3	$\rightleftharpoons$	$CaCO_3 + Na_2SO_4$	(4)

It may be noted that it is necessary to add excess lime, usually, a surplus of 35 mg/L as CaO or 1.25 mg/L above the stoichiometric requirement, to raise the pH for precipitation of magnesium by reaction at (3).

S. No.	Components	Conc. mg/L	Lime required mg/L	Soda ash required mg/L	Remarks
1	CO_2	0.45	0.45	—	Reaction (1)
2	$Ca(HCO_3)_2$	3.50	3.5	—	Reaction (2)
3	$CaSO_4$	1.50	—	1.5	Reaction (4)
4	$MgSO_4$	0.73	0.73	0.73	Reactions (3) and (4)
			4.68	2.23	

Lime required = Stoichiometric quantities + excess lime
= 4.68 + 1.25 = 5.93 mg/L
= 166.04 mg/L as CaO
= 2110.41 mg/L as $Ca(OH)_2$

Soda ash required = 2.23 mg/L
= 118.19 mg/L as Na_2C0_3

Annual consumption of lime as CaO
$= (166.04 \times 250 \times 24 \times 10^3 \times 365) / (10^6 \times 10^3)$
= **363.63** metric tons

Annual consumption of soda ash as Na_2C0_3
$= (118.19 \times 250 \times 24 \times 10^3 \times 365) / (10^6 \times 10^3)$
= **258.84** metric tons

Annexure 10.2: Design of Iron Removal Units

Typical designs of iron removal units for 5,10, and 20 m^3/h flow.

Design Considerations

- Schemes have been designed for 5, 10, and 20 m^3/h flow and 10% extra water quantity to provide for sedimentation bleed losses and filter backwash requirements.
- Power shut-downs are frequent and rarely more than two hours of supply is available in the morning and evening. Accordingly, raw water pumping hours are assumed to be two hours in the morning, and two hours in the evening. During this four hours pumping period, the total daily requirements of water are to be pumped to an elevated storage tank to draw water by gravity flow to the treatment units.
- To avoid extra cost for an additional overhead tank for filtered water, it is assumed that the filtered water from the sump well will be pumped directly for distribution. The distribution of treated water would follow the same time schedule as contemplated for pumping raw water.
- Backwashing of the sand filter would be carried out by using raw water from the overhead tank.

Design Criteria

• Water consumption	40 LPCD
• Tray aerator	
Spacing of trays	0.3 m
Aeration rate	1.26 $m^3/m^2/h$
• Sedimentation basin	
Detention period	2.5 h
• Sand filter	
Effective size	0.6 to 0.8 mm
Uniformity coefficient	1.3 to 1.7
Sand depth	1.2 m
Total head above sand	1.35 m
Rate of filtration	4.88 $m^3/m^2/h$
Minimum backwash rate	35 $m^3/m^2/h$
Total head for filter wash	12 m
Gravel depth	0.39 to 0.62 m
Gravel size	
65 to 38 mm	13 to 20 cm
38 to 20 mm	8 to 13 cm
20 to 12 mm	8 to 13 cm
12 to 5 mm	5 to 8 cm
5 to 2mm	5 to 8 cm

Specifications of units are detailed in the following table and the arrangements are as shown in:

Design Specifications for Continuous Iron Removal Unit for Community Water Supply			
Design capacity, m^3/h	5	10	20
Raw water pump, HP	2.0, 2 Nos.	3.4, 2 Nos.	7, 2 Nos.
Overhead reservoir			
Capacity, m^3	66	132	264
Size	4.7 × 4.7 × 3 m	6.2 × 6.2 × 3.5 m	8.7 × 8.7 × 3.5 m
Tray aerators			
No. of trays	4	5	6
Collection trough	1	1	1
Tray size	1.3 × 1.3	1.6 × 1.6 m	2.1 × 2.1 m
Height of each tray	0.3 m	0.3 m	0.3 m
Sedimentation tank			
Size	2.9 × 2.9 × 2.5 m	3.7 × 3.7 × 3 m	5.25 × 5.25 × 3 m
Sand filter			
Size	1.69 × 1.69 m	1.85 × 1.85 m, 2 Nos.	2.6 × 2.6 m, 2 Nos.
Clear water storage tank	4.7 × 4.7 × 3 m	6.2 × 6.2 × 3.5 m	8.7 × 8.7 × 3.5 m

Annexure 12.1: Capacity of Service Reservoir

Example 1: Find the capacity of a newly proposed service reservoir serving OZ having a demand of 2 MLD. The supply hours are 22 hours, and the supply shall shut down during the 21st and 22nd hours. The hourly multiplying factors are shown in Figure 1. The initial water elevation is 100 m and the maximum water elevation in the tank is 104 m.

It is required to compute the capacity of the service reservoir.

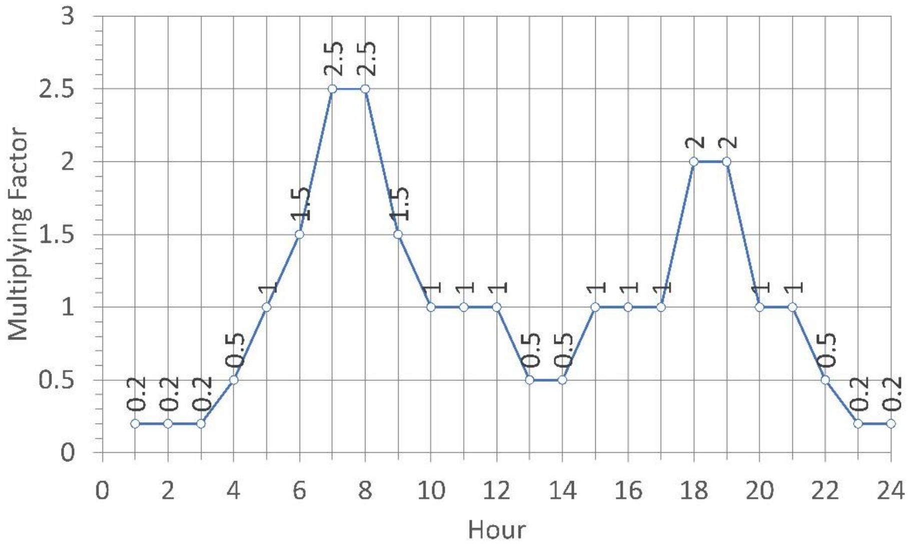


Figure 1: Multiplying factors

Solution: Average inflow = 2*1000/24= 83.33 m^3/h, since supply hours =22, Hourly inflow = 2*1000/22 = 90.909 m^3/h which are shown in Col. 2 of Table 1. Multiplying factors are shown in Column 4. Hourly out flow = Multiplying factor × Average inflow which are computed in Col 5. Difference between cumulative inflow and cumulative outflow are shown in Col. 7. The computation in Table 1 is carried out with the assumption of initial water elevation of 1.35m.

Table 1: Mass balance

Time from Start (hours)	Inflow		Multiplying factors	Outflow		Cumulative. Inflow-Cumulative. Outflow (m^3/h)
	Inflow (m^3/h)	Cumulative Inflow (m^3/h)		Outflow (m^3/h)	Cumulative Outflow (m^3/h)	
1	2	3	4	5	6	7
0	90.9	0.0	0.2	16.7	0.0	0.0
1	90.9	90.9	0.2	16.7	16.7	74.2
2	90.9	181.8	0.2	16.7	33.3	148.5
3	90.9	272.7	0.2	16.7	50.0	222.7
4	90.9	363.6	0.5	41.7	91.7	272.0
5	90.9	454.5	1	83.3	175.0	279.5
6	90.9	545.5	1.5	125.0	300.0	245.5
7	90.9	636.4	2.5	208.3	508.3	128.0
8	90.9	727.3	2.5	208.3	716.7	10.6
9	90.9	818.2	1.5	125.0	841.7	-23.5
10	90.9	909.1	1	83.3	925.0	-15.9
11	90.9	1000.0	1	83.3	1008.3	-8.3
12	90.9	1090.9	1	83.3	1091.7	-0.8
13	90.9	1181.8	0.5	41.7	1133.3	48.5
14	90.9	1272.7	0.5	41.7	1175.0	97.7
15	90.9	1363.6	1	83.3	1258.3	105.3
16	90.9	1454.5	1	83.3	1341.7	112.9
17	90.9	1545.5	1	83.3	1425.0	120.5
18	90.9	1636.4	2	166.7	1591.7	44.7
19	90.9	1727.3	2	166.7	1758.3	-31.1
20	90.9	1818.2	1	83.3	1841.7	-23.5
21	0.0	1818.2	1	83.3	1925.0	-106.8
22	0.0	1818.2	0.5	41.7	1966.7	-148.5
23	90.9	1909.1	0.2	16.7	1983.3	-74.2
24	90.9	2000.0	0.2	16.7	2000.0	0.0

The graph of Cumulative Inflow and Cumulative Outflow with time is shown in Figure 2.

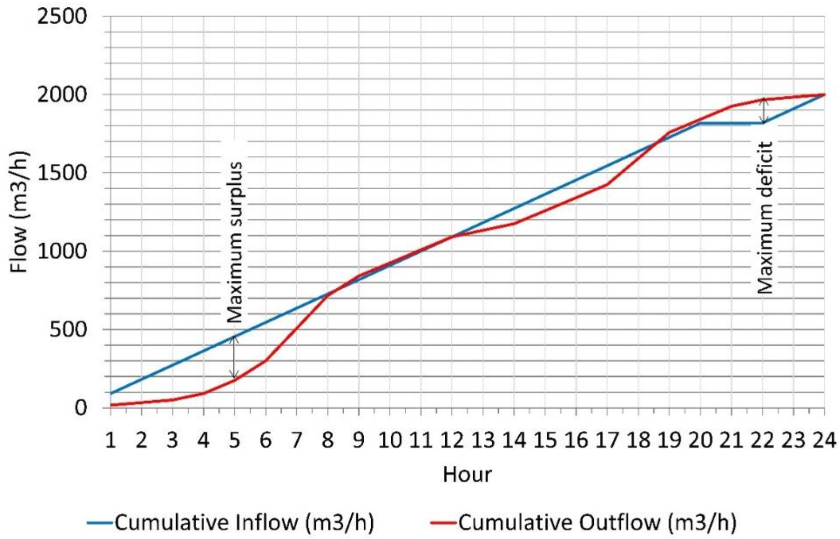


Figure 2: Graph of Cumulative Inflow and Cumulative Outflow with time

Capacity: For the assumed initial water elevation of 1.35 m, it is observed that the values of cumulative inflow are more than those of the cumulative outflow till eight hours and their difference is maximum at five hours (Figure 2). This means the surplus water is maximum at five hours. It can also be seen that reservoir will be full when the surplus is maximum and it will be empty when the deficit is maximum, which is at 22 hours.

The values of cumulative outflow are more than cumulative inflow from eight hours to 12 hours. This means that the water levels in the tank are decreasing. It may also be observed that the values of cumulative inflow are more than cumulative outflow from 12 hours till 18 hours, which means that the water level in the tank are increasing. Beyond 18 hours, cumulative outflow is more than that of the cumulative inflow.

Thus, for balancing the variation in the water levels (decreasing/ increasing), some buffer capacity should be provided which is given by,

Capacity of tank = Maximum surplus storage + Maximum deficit (Eq. 1)

The maximum surplus from Col 7 of Table1 = 279.5 and maximum deficit = 148.5 m^3. Hence,

Capacity of tank = 279.5 + 148.5 = 428 m^3.

For 24×7 water supply, the service reservoir should neither get empty nor overflow. This is achieved by properly maintaining initial water level in the tank, which is given by,

$$\textit{Initial water level in the tank} = \frac{\textit{Maximum deficit volume}}{\textit{Area of cross section}} \qquad \text{(Eq. 2)}$$

In above case, maximum deficit = 148.5 (Col 7 of Table 1), hence,

$\textit{Initial water level in the tank} = \dfrac{148.5}{0.7854(12*12)} = 1.31, \textit{Say } 1.35\ m$. Hence, assumption of initial water level of 101.35 is in order.

Capacity as per demand/3 Rule:

Demand = 2 MLD, hence the capacity = 2*1000/3 = 666.67, say 667 m^3

Though the capacity computed by the mass balance is 428 m^3, the capacity by the demand/3 rule is 667 m^3. Hence, the capacity to be adopted is 667 m^3.

Example2: Check extent of the pipe network of the existing service reservoir whose capacity is 1 ML. The pipe network of the existing tank is shown in Figure 3.

Solution:

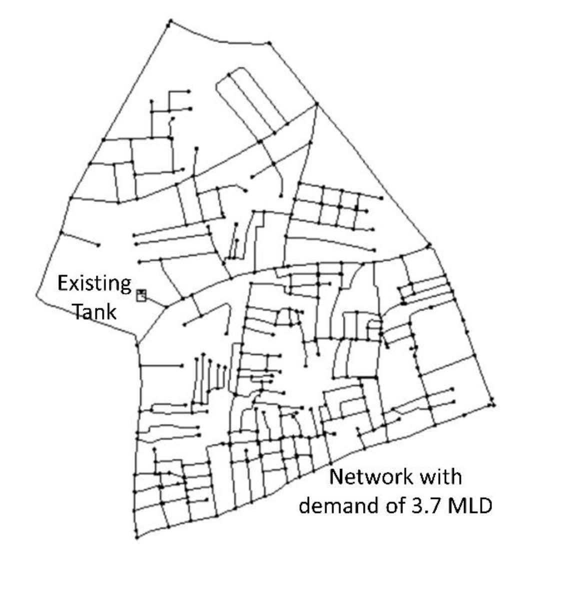


Figure 3: Existing ESR of 1 ML capacity and its OZ

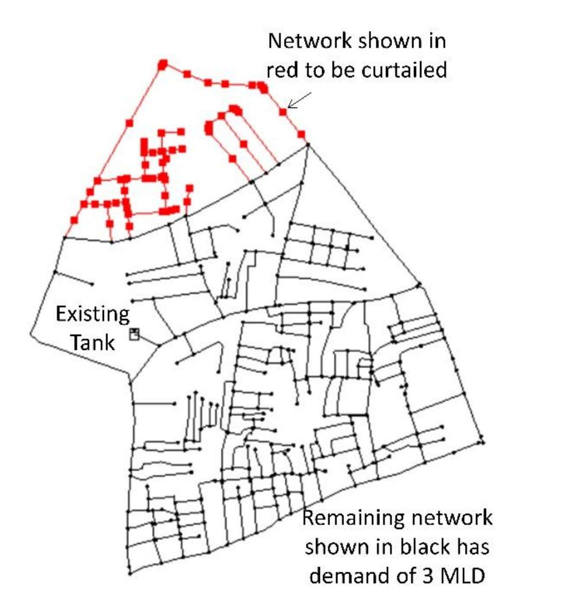


Figure 4: Curtailed network of the existing ESR of 1 ML capacity, with modified OZ

From Figure 3, it is observed that the demand of all the nodes of the existing tank is 3.7 MLD. If the network is allowed to be served by the existing tank, then the required capacity of the tank would be 3.7/3, i.e., 1.23 ML and the tank will get empty. This can be avoided by making retrofitting of the original network and some part of the network should be removed as shown in Figure 4. The removed part of this existing tank shall be then attached with the adjoining OZ of another tank.

In some of the cases, due to lesser supply hours (due to non-availability of electricity), the required capacity of the existing tank shall be computed using mass balance method and it might be larger than 33%. In such situation, the extent of the network shall be suitably curtailed/adjusted. In any case, demand of all nodes serving existing tank shall not exceed three times capacity of tank.

Annexure 12.2: Retrofitting to Refurbish Pipe Network

Retrofitting: Retrofitting means to furnish distribution pipe network with new pipes or replacement of pipes with modified diameters which are not available or considered necessary at the time of laying the network. Retrofitting of pipes is required to refurbish and reengineering the pipe network in the distribution system.

(a) Retrofitting DMAS

DMAs are the building blocks of any 24×7 water supply scheme. Most of the times the DMAs are needed to be created from the existing pipe network by retrofitting. The process is illustrated in Figures 1 and 2.

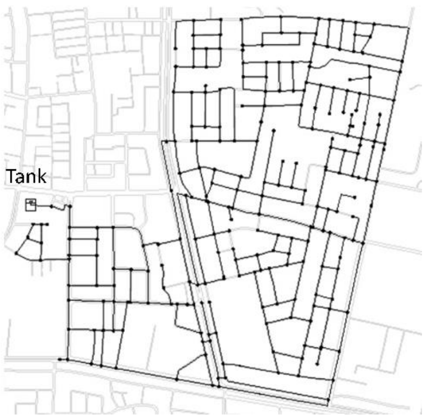
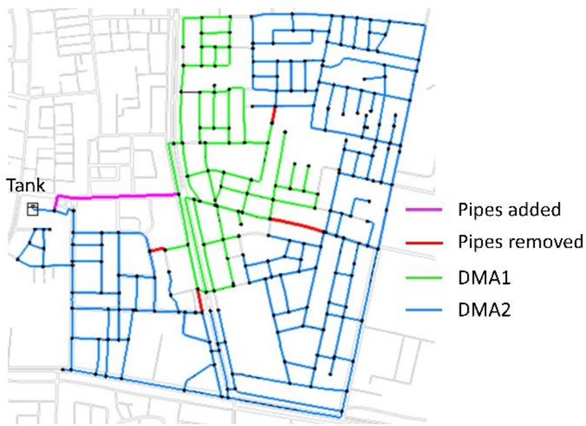
	

Figure 1: Before retrofitting	Figure 2: After retrofitting

One OZ is shown in Figure 1. It is required to divide this OZ into two DMAs. By retrofitting, some pipes shown in red are removed/ replaced to separate out the DMAs. DMA1 is shown in green colour and DMA2 in blue colour. In addition to this, one pipe from the common outlet of the service tank is added to feed DMA1 directly.

(b) Retrofitting Operational Zones (OZ)

In Chapter 2, it is mentioned that each service reservoir should have one OZ and each OZ to receive water from only single reservoir. This means OZ should be hydraulically discrete (isolated) from other OZs. The process of doing this is illustrated in Figure 3.

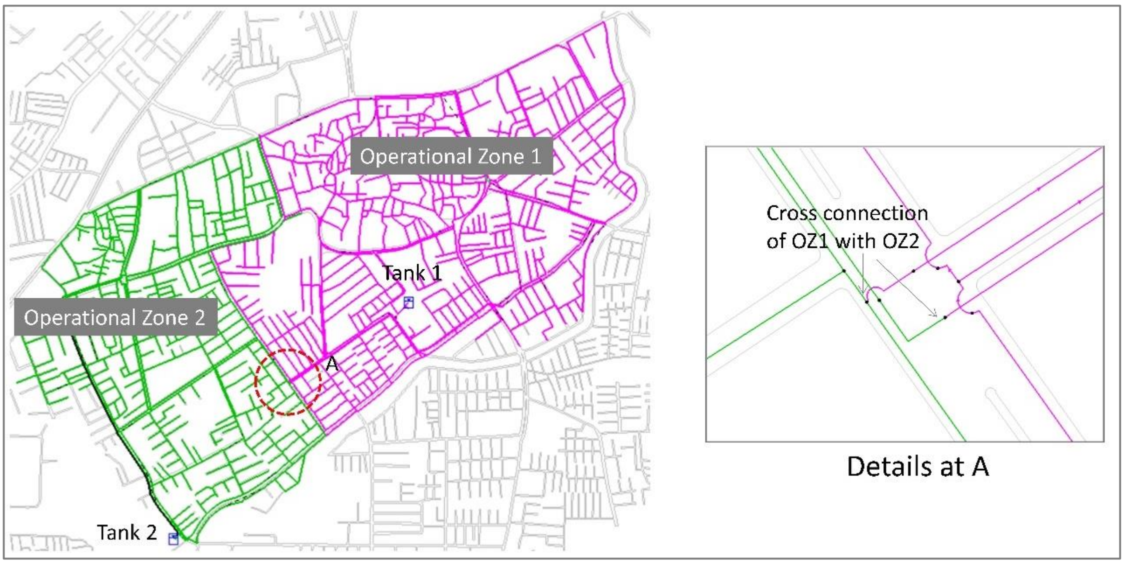


Figure 3: Removal of cross connections between OZ1 and OZ2

The OZs 1 and 2 with cross-connection between them are shown in Figure 3. These OZs should be hydraulically discrete, and hence the cross-connection should be removed.

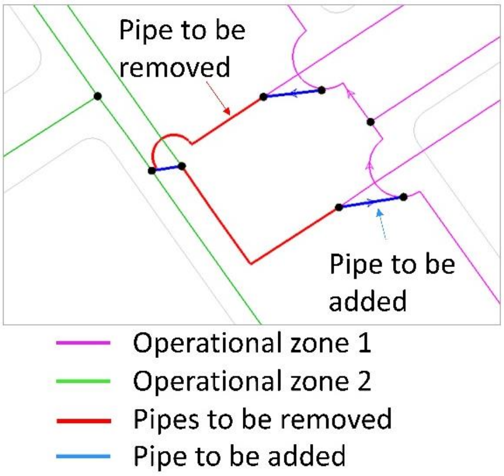


Figure 4: Before retrofitting OZs

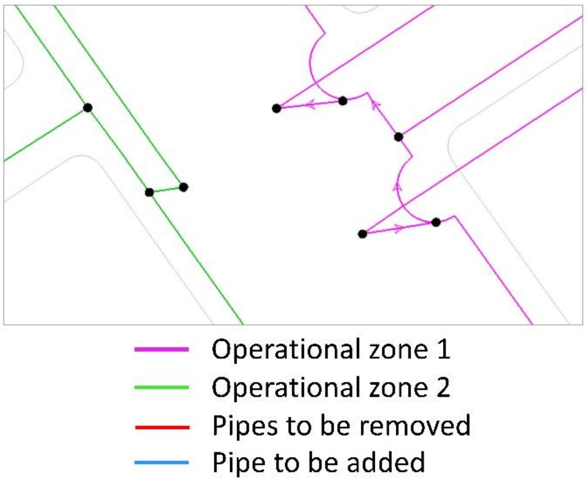


Figure 5: After retrofitting OZs

The details of pipes to be removed and added are shown in Figures 4 and 5.

(c) Retrofitting for Creating Pressure Zones

The land within the city may be uneven. Due to this, the nodal pressures are high in low lying area and low on high altitude area of the city. The OZs are selected depending on the ground elevations. Figure 6 shows such area wherein difference in elevations is more than 20 m. Therefore, it is necessary to create the pressure zones by using GIS. After forming pressure zones, the pipe network is accordingly designed. Thus, while designing OZ, creation of the pressure zones is an important task.

Figure 6 shows existing pipe network. Retrofitting of pipes is required to separate out pressure zones. Using GIS-based contours, the raster image of elevations is created

(Figure 6) and the existing network of pipes, nodes and tank are marked.

Figure 6: Creation of pressure zones

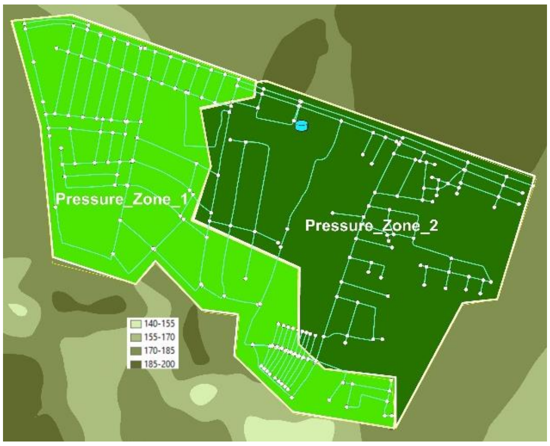


Figure 7: Marking existing pipe network in pressure zones

Here, the pressure zones are created as raster image of elevations. We need to observe which land polygon covers a part of network and accordingly, mark land polygons as shown in Figure 7.

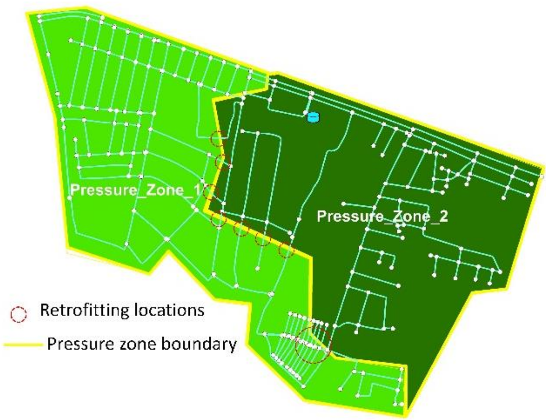


Figure 8: Retrofitting required to separate out pipe from pr. zones

Figure 9: Retrofitting required to remove the cross connections

There are number of cross connections of pipes shown under red marked circles in Figure 8. For separating the pressure zones, it is necessary to remove these cross connections as shown in Figure 9. Thus, after retrofitting the cross-connection is removed and the pipes in pressure zones 1 and 2 are separated which formed the two separate pressure zones.

Annexure 12.3: Measures to Increase Residual Pressure

The objective is to identify the areas in OZs in which the nodal pressures are less than the norm (Say, 21 m), and explain the procedure for increasing the pressure. There are four ways to increase nodal pressures:

1. Separate out part of OZ of Existing tank for nodal pressure <21m

2. Using newly proposed ESRs

3. Online boosting by VFD pump on outlet of tank for entire OZ having nodal pressures <21m

4. Online boosting by VFD pump on branch line to area having nodal pressures <21m

Part of OZ of Existing Tank: Consider one OZ as shown in Figure 1 with two DMAs. Pipes in DMA1 are shown in green colour and pipes in DMA2 are shown in blue colour.

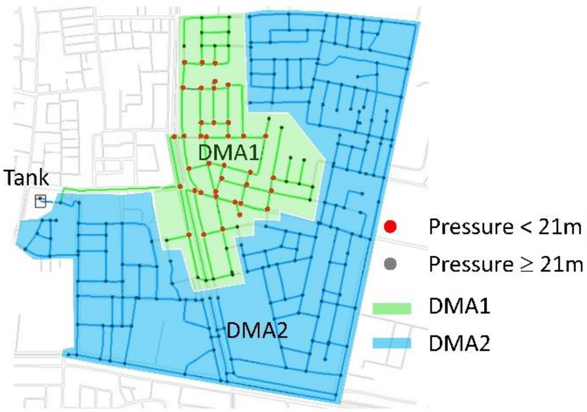


Figure 1: Nodes in DMA 1 have pressures < 21m

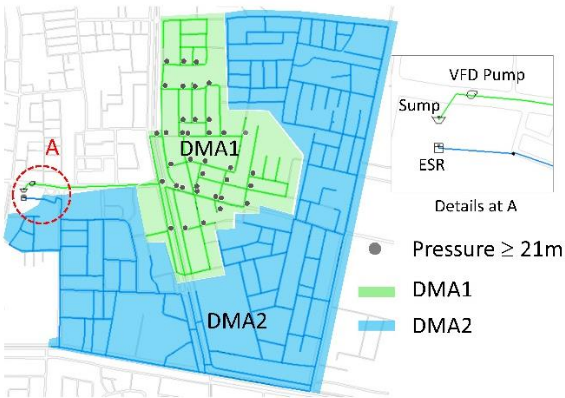


Figure 2: Direct pumping of DMA1 to increase pressures < 21m

One ESR is shown in Figure 1 which supplies water by gravity to this OZ. On running the hydraulic model, the nodal pressures are computed; some of the nodes in DMA1 have nodal pressures less than 21 m which are shown in red colour in Figure 1.

Solution: Objective is to increase nodal pressure to 21 m. As nodes in DMA1 have nodal pressures of less than 21 m, it is contemplated that water should be pumped in DMA1. The arrangement is shown in Figure 2 in which one sump is constructed near the tank. It can be seen (Figure 2) that now all the nodes in this DMA1 have pressures equal to or more than 21 m.

Thus, we can increase nodal pressures by VFD pump. Surge analysis should be carried out to avoid the negative pressures that cause cavitation.

2. Newly proposed ESRs: If the scheme is new, then new ESRs are proposed. The staging height of the newly proposed ESRs can be easily determined to get 21 m residual pressure.

3. Online Boosting on Outlet of Tank for Entire OZ: In this case, a pump is installed (Figures 3 and 4) on the outlet of the ESR.

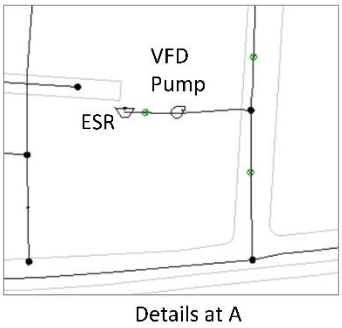


Figure 3: Network of OZ of Kotla ESR

Figure 4: Pump installed on outlet of Kotla ESR

With a small design head, nodal pressures of more than 21 m can be achieved.

4. Online boosting on branch line to area having nodal pressures <21 m

Online boosting on branch line to area (Figure 5) having nodal pressures <21 m

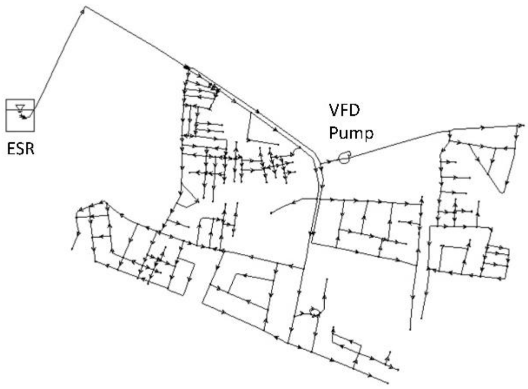


Figure 5: Area (shown in red) where pressures are less than 21 m

Figure 6: Network with VFD pump which is installed on branch pipeline which supplies water to area where pressures are less than 21 m

In this case, the VFD pump is installed on the branch (Figure 6) which supplies water to area where pressures are less than 21 m. Now the nodal pressures are >21 m.

Annexure 12.4: Hydraulic Modelling Using VFD Pump

A case study of hydraulic modelling of one OZ in Ayodhya City is discussed here. One OZ (Hanuman Garhi) of Ayodhya City is shown in Figure 1 and its network is shown in Figure 2. DMA1 is shown in green colour and DMA2 in blue colour. The existing network consists of 231 nodes and 275 pipes. The network is fed from an ESR of 12 m staging, which is found insufficient to meet the residual pressure requirement of 21 m at some of the nodes. The most critical node in the network is J-1.

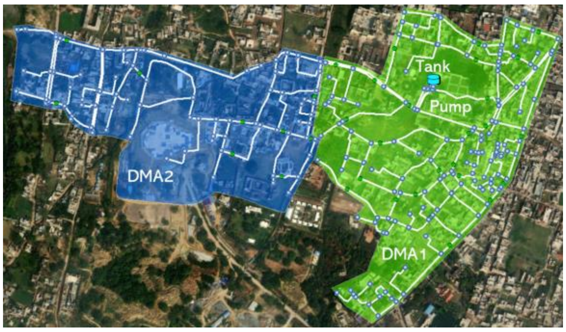


Figure 1: Network with DMAs

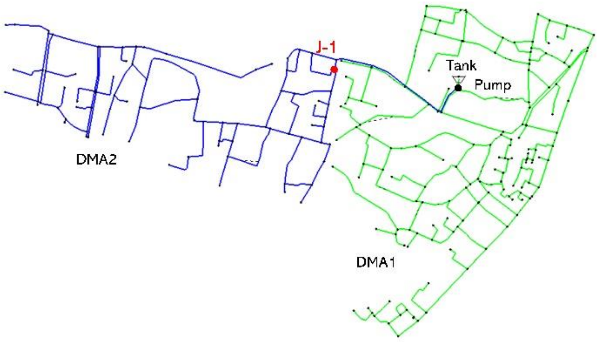


Figure 2: Network of one OZ of Ayodhya

Different options for improving network performance are considered. Hydraulic modelling is carried out for obtaining performance: (1) during period of maximum demand (peak hour) using steady state; and (2) for the entire day using extended period simulation (EPS). Available heads at the critical nodes are compared for different options.

1. MODELLING IN STEADY STATE WITH CONSTANT SPEED PUMPS

Option 1. In this option, it is assumed that existing service reservoir is discarded. A direct pumping to distribution network through constant speed pump is considered from the sump (Figure 3). The supply level in the sump is taken as 100 m.

Option 2. Reservoir is retained and an online boosting is provided by installing pump on the outlet of the tank (Figure 4). As mentioned earlier, reservoir height is 12 m, and the same is taken as supply level in steady state analysis.

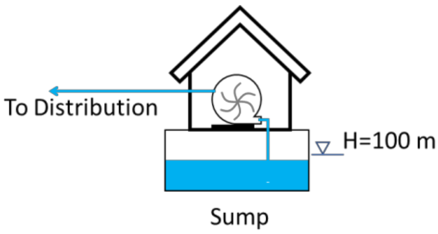


Figure 3: Option 1 - Direct pumping from the sump

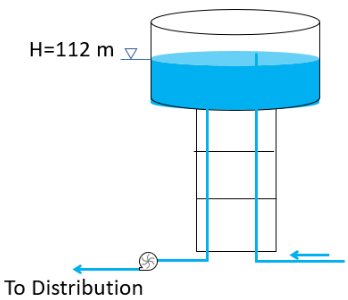


Figure 4: Option 2 - Boosting by installing pump on the outlet of the tank

The required head for the pump to meet the residual pressure requirement of 21 m at junction J-1 is obtained using steady state model for peak hour demands using one commercial network software. The minimum required head at pump is observed to be 29.6 m for the design flow of 8.788 MLD for the Option 1. In the Option 2, the design head of booster pump is observed to be reduced to 17.6 m due to positive suction head of 12 m provided by the service reservoir. Obviously, Option 2 is better than Option 1. Thus, it can be concluded that the existing ESRs, even with less staging height, cannot be the hurdle and can be very much utilised when the retrofitting exercise is taken up to increase the residual nodal pressures to 21 m. For Option 2, i.e., a flow of 8.788 MLD and 17.6m head, we need just 31 HP pump and cost of electricity (at Rs 8/kwH) would be Rs 4,434 per day.

2. EPS with constant speed pump

The model with network as shown in Figure 2 is changed to the Extended Period Simulation (EPS) by assigning the pattern of multiplier factors (Figure 5) to each of the nodes. EPS run is obtained for both the options. The available pressures at some of the nodes including that of the critical node J-1 are shown at 0 Hr and starting of the peak hour (7 Hr) are shown for the two options in Figures 6 and 7, respectively. Further, available pressures at these nodes observed during steady state analysis are shown side by side for comparison. The following can be observed from Figures 6 and 7:

1. In both the options, pressures in steady state are equal to the pressures in EPS at seven hours (peak hours). Small difference could be due to change in water level in the reservoir in the two options.
2. EPS provides available pressure more than 21 m during 0 Hr at all the nodes. A comparison of the EPS results for both the options are shown in Figure 8. Although not shown in the Figure 8, the availability of higher residual heads can be observed during non-peak hours.

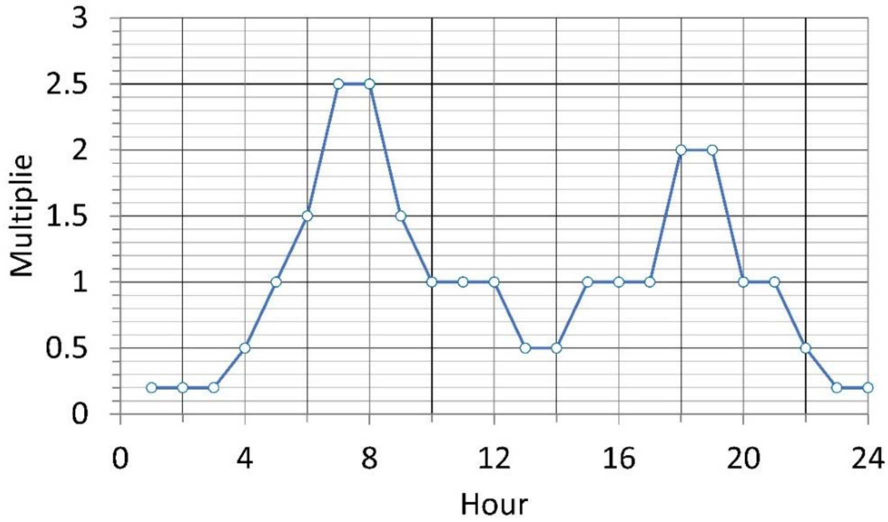


Figure 5: Peak factor

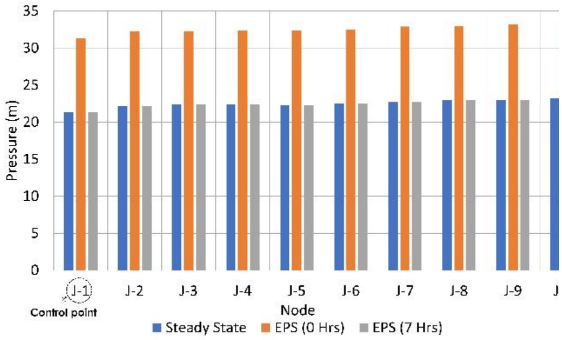


Figure 6: Option 1: Direct pumping from sump Steady State

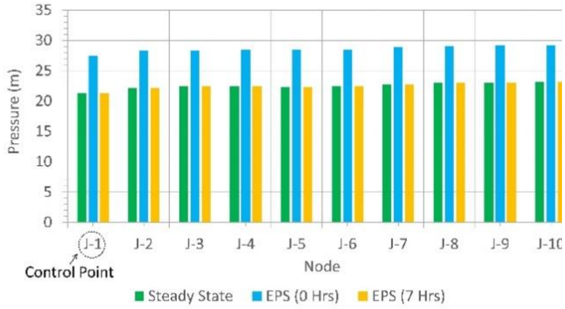


Figure 7: Option 2: Pumping by booster on outlet of tank.

As the values of available pressures at the critical node J-1 at 0-Hr and other non-peak hours are more than 21 m, it indicates over consumption of energy. Usually, in the field, in such a situation, the operating point of the pump shifts, and may go out from best operating zone, resulting in working of pump at lower efficiency and thus incur higher energy costs. Hence, VFD pumps are better option to maintain 21 m at the critical node J-1 at all the time.

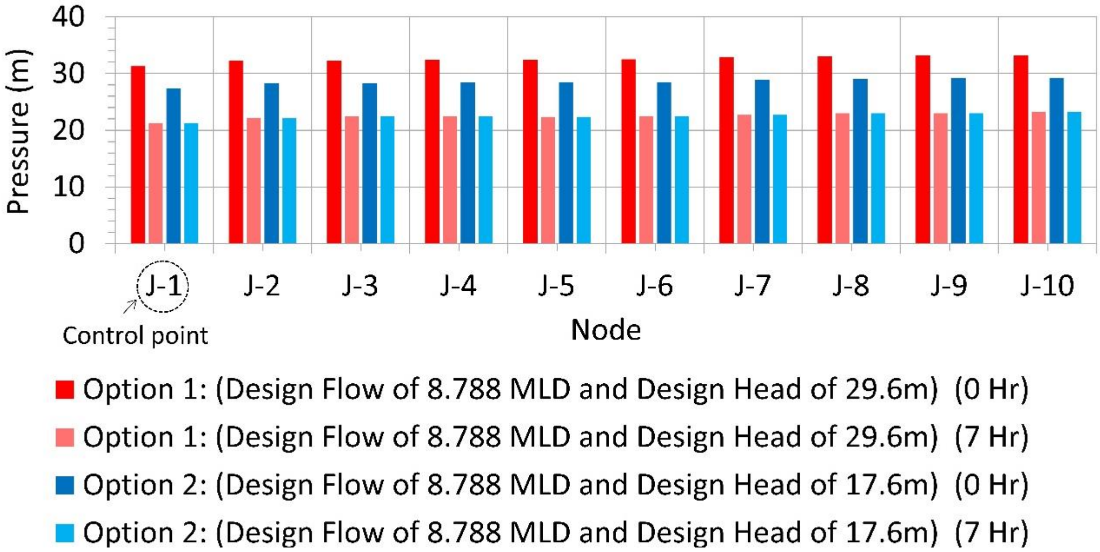


Figure 8: EPS results of pressure at nodes

3. EPS Model using VFD Operated Pump

3.1 Principle Utilised in the Software

Relative Speed Factor: In the network software, the parameter relative speed factor is used to compute the speed of pump. It is defined (Mădulărea, et al 2019) as ratio between the actual pump speed and its maximum speed. If the value of relative speed factor of the pump is calculated by the software as 0.7, it means the speed calculated is 70% of the full speed (where full speed means the pump characteristic curve as has been entered in the pump definition).

The relative speed factor is determined by how much head and the corresponding flow it takes to maintain the fixed head. It will be adjusted accordingly to maintain the pressure at the critical node. The values of relative speed factor of the pump for all the heads and flows for different points of time in a day can be calculated. Thus, if the values of relative speed factor are known, the RPM of the pump can be known and by feeding them to the actuator, the actual pump speed can be adjusted.

Hence, the objective of running the model is to compute the relative speed factors at different point of times in a day to maintain the 21 m head at the critical node.

3.2 Running Model with VFD Pump

Option 1 (Direct pumping from the sump): The value of the relative speed factor of the VFD pump is set as 1. With an elevation of 100 m in the tank, the model is run in EPS with the VFD operated pump. The model results of few nodes are shown in Figure 9.

Option 2 (Boosting by installing pump on the outlet of the tank): With an elevation of 112 m in the tank, the model is run in EPS with the VFD operated pump. The model is run for the same value of the relative speed factor of 1 and the model is run. Results of the few nodes are shown in Figure 9.

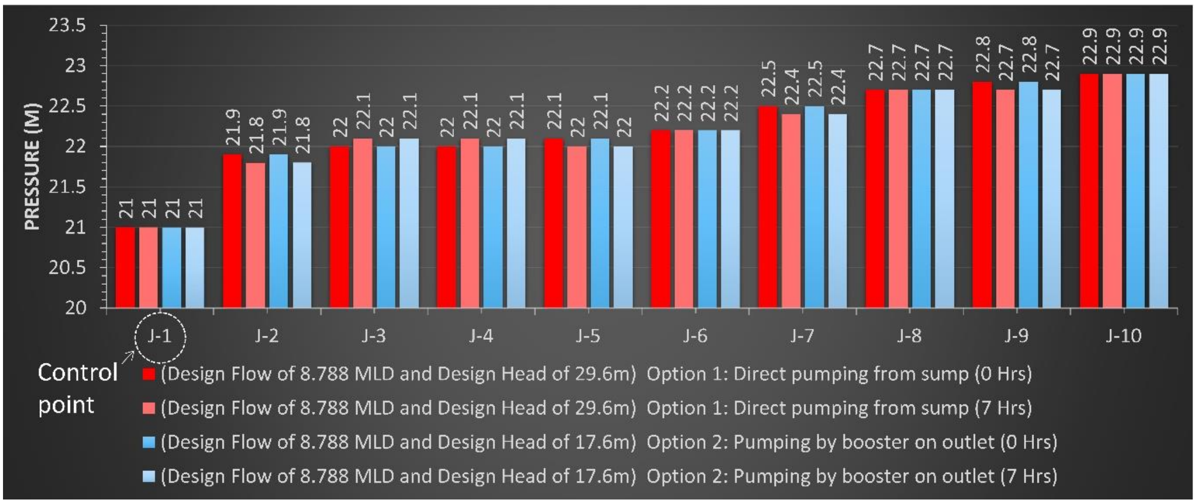


Figure 9: Pressure at nodes

From Figure 9, it can be observed that the values of residual pressure (m) at the critical node (J-1) are now exactly equal to 21 m in both Options 1 and 2 and for zero and seven hours. In fact, the same is also observed at other non-peak hours. This is due to reduction in motor RPM. The relative speed factors of the pump at various time are shown in Table 1.

Table 1: Relative speed factors of the VFD pump

Time (hours)	Option 1: Direct pumping from the sump	Option 2: Boosting by installing pump on the outlet of the tank
	(Design Flow of 8.788 MLD and Design Head of 29.6 m)	(Design Flow of 8.788 MLD and Design Head of 17.6 m)
0	0.859	0.853
1	0.859	0.853
2	0.859	0.853
3	0.859	0.853
4	0.864	0.858
5	0.881	0.876
6	0.91	0.905
7	0.996	0.993
8	0.996	0.993
9	0.91	0.905
10	0.881	0.876
11	0.881	0.876
12	0.881	0.876
13	0.864	0.858
14	0.864	0.858
15	0.881	0.876
16	0.881	0.876
17	0.881	0.876
18	0.948	0.945
19	0.948	0.945

Time (hours)	Option 1: Direct pumping from the sump	Option 2: Boosting by installing pump on the outlet of the tank
	(Design Flow of 8.788 MLD and Design Head of 29.6 m)	(Design Flow of 8.788 MLD and Design Head of 17.6 m)
20	0.881	0.876
21	0.881	0.876
22	0.864	0.858
23	0.859	0.853
24	0.859	0.853

Thus, VFD controller adjusts the RPM of motor as per the requirement. Depending upon the requirement, VFD controlled pump can be designed. Obviously, VFD pumps are costly as compared to constant speed pumps. However, they result in lowing of energy consumption and prove to be economical in the long run.

5. Conclusions:

The main advantage of 24×7 water supply is that there is no chance of contamination from outside as the pipelines are always pressurised. Further, it is necessary that water is available to consumer at tap. Considering the same, the CPHEEO in recent guidelines increased the residual pressure requirement at the service point of consumer to 15-21 m. Maintaining high residual pressure may require need of direct pumping either using constant speed pumps or variable flow drive pumps.

The network of the OZ (Hanuman Garhi) of Ayodhya City has been modelled in one commercial software using both types of pumps. For maintaining the 21 m at critical node, the software computed the values of the relative speed factors of the VFD operated pump. After knowing the relative speed factors of the VFD operated pump, the pump curves from the manufactures' catalogue can be selected. In this way, a single pump with same impeller can deal with change in demand in non-peak and peak hours. In case variation in minimum and maximum discharge through VFD pump is large, a floating reservoir at suitable locations on downstream of network can be tried to reduce the difference between maximum and minimum flow through VFD.

Smart pump control philosophy can be implemented to regulate the pump's operation at variable speed, meeting varying demand throughout the day as well as maintaining required minimum residual pressure at critical point in the distribution network. This also reduces energy consumption and pressure fluctuations in the system.

It is recommended to adopt this method in metro cities where the distribution network of proper pipe materials is available, and electricity is continuously available through express feeders.

References:

1. Addendum, 2023, retrieved from https:// mohua.gov.in/ publication.php? sa= manuals-and-advisories.php
2. Madularea R. A., Georgescu P. O. and Georgescu, S. C. (2019). "Speed factors computed for pumping schedules in Water Distribution Networks: DDA versus PDA formulations," E3S Web of Conferences 85, 06002 (2019) EENVIRO 2018 retrieved from https://www.e3s-conferences.org/articles.
3. Walski et al. (2010), "Developing System Head Curves for Closed System," Journal AWWA, September 2010

Annexure 12.5: Method of Network Analysis

(a) Hardy Cross Method

(i) Balancing Heads

In this method, from the knowledge of system inflows and outflows, the flows in all the pipes of the network are distributed to meet continuity constraints at all the nodes. When inflows and outflows are explicitly known, this will involve initial assigning of flows in one of the pipes of every primary loop in the system. Based on this assignment, flows in other pipes of the loops are assigned. The requirement that the sum of head losses around all primary loops should equal zero gives rise to a system of as many equations as number of loops. The requirement of total head loss between source nodes is satisfied by considering additional pseudo-pipe (an imaginary infinite resistance pipe connecting the two source nodes to form a pseudo-loops) Solution of the exactly determined system of non-linear equations is affected by a systematic relaxation in the Hardy Cross method. In the Hardy Cross method of balancing heads, which is a trial-and-error process, the correction factor for assumed flows (necessary formulae are made algebraically consistent by arbitrarily assigning positive signs to clockwise flows and associated head losses and negative signs to anti-clockwise flows and associated head losses) □Q in a circuit is calculated by the formula:

$$\Delta Q = \frac{-\sum h}{n.\sum h/Q} \qquad \text{(Eq. 12.1a)}$$

where ΔQ= is loop flow correction.

The assumed flows are corrected accordingly, and the procedure is repeated until the required degree of precision is reached. This is essentially a repetitive procedure. The sequential steps are presented below:

a) Assume suitable values of flow Q in each pipeline such that the flows coming into each junction of the loop are equal to flows leaving the junction.

b) Assign positive sign to all clockwise flows and negative sign to all anti-clockwise flow.

c) Compute the head loss H in each pipe by use of the friction formula.

d) Compute ∑h (i.e., algebraic sum of the head losses) around each loop and if this is nearly equal to zero in all loops (within allowable limits of ±0.01 m), the assumed flows are correct.

e) Otherwise, if ∑h is not equal to 0 for any loop, compute the error in loop flow using Eq. 12.1a, for real as well as pseudo-loops.

f) Pipes operating in more than one circuit draw corrections from each circuit. However, the second correction is of the opposite sign as applied to the first circuit.

g) Repeat the cycle, till ∑h (around each loop) is nearly equal to zero within the allowable limits. Then, the final values of flows are the actual values in the pipelines.

(ii) Balancing Flows

When using the method of balancing flows at junctions or nodes of the system, pressures at nodes are assumed on the basis of given pressure surface elevations at some nodes (e.g., fixed elevation reservoirs) and the flows in the pipes are estimated.

In the method of balancing flows (modification of original Hardy Cross Method), which is applicable to junctions and nodes, the flows at each junction are made to balance for the assumed heads at the junctions and the corresponding head losses in the pipes. The correction factor for assumed head losses in the pipes is calculated using the formula:

$$\Delta H = \frac{\sum Q}{\sum Q/_{n\,h}} \qquad \text{Eq. (12.2a)}$$

The steps in the computation are as under:

(i) Assume heads at all the free junctions.

(ii) Assign positive sign to head losses for flows towards the junction and negative sign to those away from the junction.

(iii) Compute the flows in each pipe by use of the friction formula.

(iv) Compute ∑Q (i.e., algebraic sum of the flows) at each fret junction and if this is nearly equal to zero at all junctions (within allowable limits of ±2%), the assumed head losses are correct.

(v) Otherwise, if ∑Q is not equal to zero at any junction, compute the correction in head by using the Eq. (12.2a).

(vi) Add the correction factor to the assumed heads with due regard to the sign of corrections.

(vii) Pipes common to more than one node receive corrections from each node.

(viii) Repeat the cycle till ∑Q= 0 at each node or junction when the final corrected values of H are obtained.

The Hardy Cross method considers one equation at a time to obtain corrections to the assumed link flows (method of balancing heads) or assumed nodal heads (method of balancing flows). The method is good for hand calculations as only one equation is considered at a time for solving. However, it is slow converging and may sometimes diverge.

(b) Newton-Raphson Method

Network balancing using Newton-Raphson method is again an iterative process, but the method seems to be faster and convergence much more rapid from a reasonably good start. The principle of this method is explained most simply by reference to solution of a single equation f(p) = 0. According to Newton's rule, if p is an approximation to a root of f(p), then (p + Δp) is a better approximation where:

$$\Delta p = \frac{f(p)}{f\prime(p)} \qquad \text{Eq. (12.3a)}$$

The nature of this result can be recognised from the Taylor series expansion of f(p+Δp), viz.

$f(p + \Delta p) = f(p) + \Delta p . f''(p) + + \cdots +$ terms involving higher powers of (Δp) Eq. (12.4a)

f(p + Δp) is equal to zero, if (p + Δp) is in reality a solution to f(p)= 0. If, in the above equation, the terms involving powers of Δp higher than the first are neglected, one obtains Newton's rule. The method can be extended to the solution of n simultaneous equations with n variables.

In setting up a water distribution network for balancing heads by Newton-Raphson method on the computer, it is useful to note the following steps and observations. Flows in the pipes are assumed to meet all the continuity constraints. The flows in all pipes of loop i are assumed to be in error by ΔQ_i; correction from both loops, the one coming from the loop under consideration being algebraically added, the other being algebraically deducted.

Equations to balance head losses around loops are then framed in terms of corrected flows. These equations are solved simultaneously to obtain loop flow corrections for all loops (both real and pseudo). The computed ΔQi, are applied to all pipes of the network as explained under Hardy Cross method giving due consideration to common pipes between loops and the iteration proceeds. The programme terminates at the allowable head tolerance or when iterations exceed a certain prescribed limit.

A general computer programme for network head balance according to Newton-Raphson method is required to compute from input values and set up the coefficient matrix for solution for ΔQ_i's. The set of linear simultaneous equations could be solved by calling appropriate library subroutines. The success of the Newton-Raphson technique lies in the selection of a good starting approximation. If the approximation is poor, it can result in the divergence of the solution. Computer programmes are readily available for the Newton-Raphson technique. The method can be applied to balance flows by assuming nodal heads also as in Hardy Cross Method.

(c) Linear Theory Method

This method, proposed by Wood and Charles, is useful for network balancing through "balancing heads by correcting assumed flows". This is also an iterative method, said to converge faster than the Hardy Cross method.

In the methods of balancing described earlier, it is necessary to assume certain values for the variables to start the iterative procedure. Naturally, therefore, the number of iterations depend upon the initial guess. No such initialisation is needed in the linear theory method.

The linear theory transforms the loop head loss non-linear relationships into linear relationships by approximating the head loss in each pipe by

$$h_x = (RQ^n)_x = (RQ^{n-1})_P Q_x = (R'Q)_x \qquad \text{Eq. (12.5a)}$$

in which Q_x is the assumed flow in pipe x. Thus, the pipe resistance constant R_x is replaced by $(R')_x$ so that, $(R')_x = (RQ^{n-1})$

All the non-linear loop head loss relationships become linear. These linear equations and the node flow continuity linear equations are solved simultaneously to obtain all Q_x values. The solution, however, will not be correct as the obtained Q_x values will not be the same as assumed Q_x values. However, it is claimed that by repeating the process several times, the obtained and the assumed values will be found to be identical, thus giving the correct solution.

In the linear theory, for the first iteration, all the Q_x values are taken as 1 giving R′ = R. It is observed that in this method, if used just as suggested earlier, yields pipe flows which tend to oscillate about the final solution. To obviate this, Wood and Charles have suggested that after two iterative solutions, for all the iterations thereafter, the initial flow rates to be used in the computations should be the average of the flow rates obtained from the past two iterations. Better would be to take the average of assumed and obtained values of previous iteration (Bhave et al, 2006). Thus, for the i[th] iteration,

$$ {}_{i+1}Q_{xa} = \frac{{}_iQ_{xa} + {}_iQxo}{2} \qquad \text{Eq. (12.6a)} $$

in which the subscript i, i+1 denotes two successive iterations. Q_{xa} and Q_{xo} are the assumed and obtained values of Q in ant iteration.

The Newton-Raphson and Linear Theory methods, linearises the non-linear equations. While Newton-Raphson method uses truncated Taylor’s series to obtain corrections to assumed pipe discharges or nodal heads successively till convergence, the Linear Theory method merges the non-linear part with resistance to linearise them and upgrade the assumed pipe discharges or nodal heads till they stabilise. Convergence in both the methods is fast. Linear theory method does not require initialisation and the iterative procedure can be started with the same values of pipe discharges in each link.

(d) Gradient Method

The gradient method solves Q-H equations by simultaneously solving set of Eqs. (12.1) and (12.2). The upgraded values of ${}_{t+1}Q$ and ${}_{t+1}H$ for any t+1 iteration can be obtained from the known values of ${}_tQ_{ox}$, ${}_tH_{oi}$ and ${}_tH_{oj}$ in t[th] iteration as described below. Let ${}_t\Delta Hi$, ${}_t\Delta Hj$, and ${}_t\Delta Qx$ be the unknown corrections for the t[th] iteration. The pipe head loss equation with truncated Taylor’s series expansion of non-linear term can be written as

$$ \left({}_tH_{oi}+{}_t\Delta H_i\right)-\left({}_tH_{oj}+{}_t\Delta H_j\right)=R_{ox}\;{}_tQ_{ox}^n + nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}{}_t\Delta Q_x, x=1,\ldots,X \qquad \text{(Eq. 12.7a)} $$

in which R_{ox} = known resistance constant of pipe *x*.

Rewriting Eq. (12.11) by transferring fixed nodal head terms, if any, on the right-hand side and term containing ΔQ_x on the left-hand side

$$ {}_{t+1}H_i - {}_{t+1}H_j - nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}{}_t\Delta Q_x = R_{ox}\;{}_tQ_{ox}^n, x=1,\ldots,X \qquad \text{(Eq. 12.8a)} $$

Subtracting $nR_{ox}\;{}_tQ_{ox}^n$ from either side

$$ {}_{t+1}H_i - {}_{t+1}H_j - nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}\left({}_tQ_{ox}+{}_t\Delta Q_x\right) = (1-n)R_{ox}\;{}_tQ_{ox}^n, x=1,\ldots,X \qquad \text{(Eq. 12.9a)} $$

Replacing ${}_tQ_{ox}$ + ${}_t\Delta Q_x$ by ${}_{t+1}Q_x$

$$ {}_{t+1}H_i - {}_{t+1}H_j - \left(nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}\right){}_{t+1}Q_x = (1-n)R_{ox}\;{}_tQ_{ox}^n, x=1,\ldots,X \qquad \text{(Eq. 12.10a)} $$

Equation (12.10a) provides *X* number of linearised equations involving corrected values of pipe discharges and nodal heads as unknowns.

Linear node flow continuity equations can be written for corrected discharge values as

$$\sum_{x \text{ connected to } j} {}_{t+1}Q_x + q_{oj} = 0, j = M+1,\ldots,M+N \quad \text{(Eq.12.11a)}$$

which are *N* linear equations. Solution of set of Eqs. (12.14) and (12.15) provides the corrected values of *X* pipe discharges and *N* unknown nodal heads.

Pipe discharge ${}_tQ_{ox}$ can be taken as unity for the first iteration or can be alternatively taken as some other arbitrarily chosen value.

Todini and Pilati (1987) suggested matrix form of Gradient method and showed that by starting with unit flow in all pipes, improved values of nodal heads and nodal flows can be iteratively obtained by solving the following equations in the matrix form. The iterative procedure can be terminated when no or negligible change in the values in two successive iterations are obtained.

The improved values of nodal heads **H** in matrix form are given by:

$${}_{t+1}\mathbf{H} = -[\mathbf{A_{21}}(\mathbf{NA_{11}})^{-1}\,\mathbf{A_{12}}]^{-1}[\mathbf{A_{21}}(\mathbf{NA_{11}})^{-1}(\mathbf{A_{11}}\,{}_t\mathbf{Q} + \mathbf{A_{10}}\,\mathbf{H_0})-(\mathbf{A_{21}}\,{}_t\mathbf{Q} - \mathbf{q_0})] \quad \text{Eq. (12.12a)}$$

$${}_{t+1}\mathbf{Q} = (\mathbf{I} - \mathbf{N}^{-1})\,{}_t\mathbf{Q} - [\,\mathbf{N}^{-1}\,\mathbf{A_{11}}^{-1}\,(\mathbf{A_{12}}\,{}_{t+1}\mathbf{H} + \mathbf{A_{10}}\,\mathbf{H_0})] \quad \text{Eq. (12.13a)}$$

In which **N** and $\mathbf{A_{11}}$ are diagonal matrix of size (X, X); $\mathbf{A_{12}} = \mathbf{A_{21}}^{\mathbf{T}}$ is unknown-head node incidence matrix of size (X, N); $\mathbf{A_{22}}$ = 0, a small null matrix of size (N, N); **Q** is column matrix of unknown pipe discharges of size (X, 1); **H** is column matrix of unknown nodal heads of size (N, 1); $\mathbf{A_{10}}$ is known head node incidence matrix of size (X, M); $\mathbf{H_0}$ is column matrix of known nodal heads of size (M, 1); **Q** is column matrix of known or assumed pipe discharges of size (X, 1); q_0 is column matrix of known nodal demands of size (N, 1); I is an identity matrix.

The gradient method is basically an application of Newton-Raphson method to simultaneously obtain unknown Q and H. The gradient method is also fast converging and like Linear Theory method, it does not require initialisation. The hydraulic solver EPANET is based on Gradient Method (Rossman 2001). Formulation and solution using matrix method is explained with a simple example. Consider a two-source network as shown in Figure 1. The network has three demand nodes - 3, 4, and 5. Total number of pipes are six.

The HGL at the source nodes 1 and 2 are: H_1 =100 m, and H_2 = 95 m. The length of pipes 1 to 6 are: L_1= 200 m, L_2 = 200 m, L_3=200 m, L_4 = 200 m, L_5 = 300 m, and L_6 = 300 m. The pipe diameters for pipes 1 to 6 are: D_1 = 250 mm, D_2 = 300 mm, D_3 = 300 mm, D_4 = 250 mm, D_5=250 mm, and D_6 = 250 mm. The H-W constant for all the pipes is 100. The nodal demands at nodes 3 to 5 are q_3 = 0.3 m^3/s, q_4 = 0.2 m^3/s, and q_5 = 0.1 m^3/s.

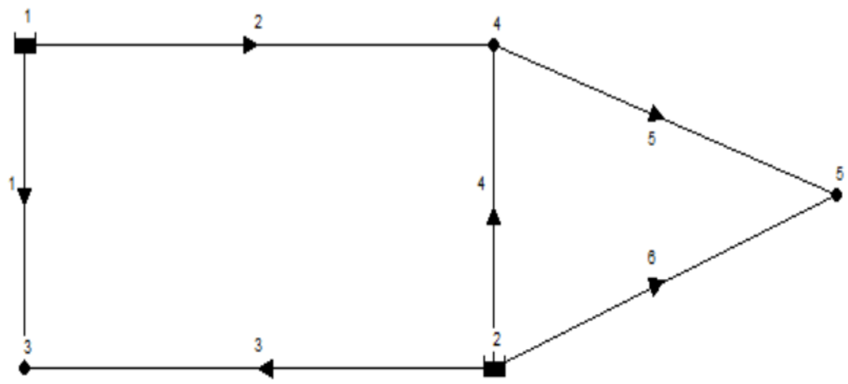


Fig. 1 Two-source, three-demand-node looped network

For the network, the number of pipes, X=6; the number of source nodes, M=2; and the number of demand nodes, N=3. Let the assumed values of pipe discharges for the first iteration be Q_1, …, Q_6 = 1.

The matrix method alternatively determines improved values of heads and flows using the following formulation:

$$_{t+1}\mathbf{H} = -\mathbf{A_{21}}\ (\mathbf{NA_{11}})^{-1}\ \mathbf{A_{12}})^{-1}\ [\mathbf{A_{21}}\ (\mathbf{NA_{11}})^{-1}\ (\mathbf{A_{11}Q} + \mathbf{A_{10}}\ \mathbf{H_0}) - (\mathbf{A_{21}Q} - \mathbf{q_0})]$$

$$_{t+1}\mathbf{Q} = (\mathbf{I} - (\mathbf{NA_{11}})^{-1}\ \mathbf{A_{11}})\ \mathbf{Q} - (\mathbf{NA_{11}})^{-1}\ (\mathbf{A_{12}}\ _{t+1}\mathbf{H} + \mathbf{A_{10}}\ \mathbf{H_0})$$

Matrices $\mathbf{A_{12}}$, $\mathbf{A_{10}}$, **Q** and **R** for the network are given by

$$\mathbf{A_{12}}=\begin{bmatrix}1 & 0 & 0\\ 0 & 1 & 0\\ 1 & 0 & 0\\ 0 & 1 & 0\\ 0 & -1 & 1\\ 0 & 0 & 1\end{bmatrix};\quad \mathbf{A_{10}}=\begin{bmatrix}-1 & 0\\ -1 & 0\\ 0 & -1\\ 0 & -1\\ 0 & 0\\ 0 & -1\end{bmatrix};\quad \mathbf{Q}=\begin{bmatrix}1\\1\\1\\1\\1\\1\end{bmatrix};\quad \mathbf{R}=\begin{bmatrix}257.626\\ 106.0174\\ 106.0174\\ 257.626\\ 386.439\\ 386.439\end{bmatrix}$$

Matrices $\mathbf{A_{12}}$ and $\mathbf{A_{10}}$ shows connectivity between nodes and pipes. Rows belongs to pipes and columns belongs to nodes. In column 1 of $\mathbf{A_{12}}$, connectivity at node 3 is shown. Pipes 1 and 3 are marked as 1 and others as 0. Both 1 and 3 are marked as positive as both are supply pipes at node 3. In case of outgoing pipe, pipe is marked negative. **Q** is column matrix with initial assumed value as 1. Pipe resistances are calculated, and matric **R** is framed.

Diagonal matrix A_{11}, in which diagonal term is $R_x|Q|^{n-1}$, is:

$$\mathbf{A_{11}} = \begin{bmatrix} 257.626 & 0 & 0 & 0 & 0 & 0 \\ 0 & 106.0174 & 0 & 0 & 0 & 0 \\ 0 & 0 & 106.0174 & 0 & 0 & 0 \\ 0 & 0 & 0 & 257.626 & 0 & 0 \\ 0 & 0 & 0 & 0 & 386.439 & 0 \\ 0 & 0 & 0 & 0 & 0 & 386.439 \end{bmatrix}$$

Matrix $\mathbf{A_{21}}$, which is transpose of matrix $\mathbf{A_{12}}$, is

$$\mathbf{A_{21}} = \begin{bmatrix} 1 & 0 & 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 1 & -1 & 0 \\ 0 & 0 & 0 & 0 & 1 & 1 \end{bmatrix}$$

Diagonal matrix **N**, in which the diagonal term is n is

$$\mathbf{N} = \begin{bmatrix} 1.852 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1.852 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1.852 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1.852 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1.852 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1.852 \end{bmatrix}$$

Matrix product $\mathbf{NA_{11}}$ is

$$\mathbf{NA_{11}} = \begin{bmatrix} 477.1234 & 0 & 0 & 0 & 0 & 0 \\ 0 & 196.3442 & 0 & 0 & 0 & 0 \\ 0 & 0 & 196.3442 & 0 & 0 & 0 \\ 0 & 0 & 0 & 477.1234 & 0 & 0 \\ 0 & 0 & 0 & 0 & 715.6851 & 0 \\ 0 & 0 & 0 & 0 & 0 & 715.6851 \end{bmatrix}$$

Inverse of $\mathbf{(NA_{11})}$ is

$$\mathbf{(NA_{11})^{-1}} = \begin{bmatrix} 0.002096 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.005093 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.005093 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.002096 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.001397 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.001397 \end{bmatrix}$$

Matrix product $\mathbf{A_{21}\,(NA_{11})^{-1}}$ is,

$$\mathbf{A_{21}\,(NA_{11})^{-1}} = \begin{bmatrix} 0.002096 & 0 & 0.005093 & 0 & 0 & 0 \\ 0 & 0.005093 & 0 & 0.002096 & -0.0014 & 0 \\ 0 & 0 & 0 & 0 & 0.001397 & 0.001397 \end{bmatrix}$$

Matrix $\mathbf{A_{21}\,(NA_{11})^{-1}\,A_{12}}$ is,

$$\mathbf{A_{21}\ (NA_{11})^{-1}\ A_{12}} = \begin{bmatrix} 0.007189 & 0 & 0 \\ 0 & 0.008586 & -0.0014 \\ 0 & -0.0014 & 0.002795 \end{bmatrix}$$

Matrix $\mathbf{-\ (A_{21}\ (NA_{11})^{-1}\ A_{12})^{-1}}$ is given by,

$$\mathbf{-\ (A_{21}\ (NA_{11})^{-1}\ A_{12})^{-1}} = \begin{bmatrix} -139.102 & 0 & 0 \\ 0 & -126.781 & -63.3905 \\ 0 & -63.3905 & 389.538 \end{bmatrix}$$

Matrices $\mathbf{A_{11}Q}$, $\mathbf{A_{10}\ H_0}$ and their addition $\mathbf{A_{11}Q + A_{10}\ H_0}$ are given by

$$\mathbf{A_{11}Q} = \begin{bmatrix} 257.626 \\ 106.0174 \\ 106.0174 \\ 257.626 \\ 386.439 \\ 386.439 \end{bmatrix} ;\ \mathbf{A_{10}\ H_0} = \begin{bmatrix} -100 \\ -100 \\ -95 \\ -95 \\ 0 \\ -95 \end{bmatrix} ;\ \mathbf{A_{11}Q + A_{10}\ H_0} = \begin{bmatrix} 157.626 \\ 6.017381 \\ 11.01738 \\ 162.626 \\ 386.439 \\ 291.439 \end{bmatrix}$$

Matrix $\mathbf{A_{21}\ (NA_{11})^{-1}\ (A_{11}Q + A_{10}\ H_0)}$ is given by,

$$\mathbf{A_{21}\ (NA_{11})^{-1}\ (A_{11}Q + A_{10}\ H_0)} = \begin{bmatrix} 0.38648 \\ -0.16846 \\ 0.947174 \end{bmatrix}$$

Matrices $\mathbf{A_{21}Q}$ and $\mathbf{A_{21}Q - q_0}$ are given by

$$\mathbf{A_{21}Q} = \begin{bmatrix} 2 \\ 1 \\ 2 \end{bmatrix} ;\ \mathbf{A_{21}Q - q_0} = \begin{bmatrix} 1.7 \\ 0.8 \\ 1.9 \end{bmatrix}$$

Matrix $\mathbf{[A_{21}\ (NA_{11})^{-1}\ (A_{11}Q + A_{10}\ H_0) - (A_{21}Q - q_0)]}$ is given by,

$$\mathbf{[A_{21}\ (NA_{11})^{-1}\ (A_{11}Q + A_{10}\ H_0) - (A_{21}Q - q_0)]} = \begin{bmatrix} -1.31352 \\ -0.96846 \\ 0.95283 \end{bmatrix}$$

Now, matrix $\mathbf{{}_{t+1}H}$ is obtained as

$$\mathbf{{}_{t+1}H = -A_{21}\ (NA_{11})^{-1}\ A_{12})^{-1}\ [A_{21}\ (NA_{11})^{-1}\ (A_{11}Q + A_{10}\ H_0) - (A_{21}Q - q_0)]}$$

$$\mathbf{{}_{t+1}H} = \begin{bmatrix} 182.7127 \\ 183.1827 \\ 432.5532 \end{bmatrix}$$

To find ${}_{t+1}\mathbf{Q}$ following step by step analysis, the following matrices are obtained.

$$\mathbf{A_{12\ t+1}\ H} = \begin{bmatrix} 182.7127 \\ 183.1827 \\ 182.7127 \\ 183.1827 \\ 249.3705 \\ 432.5532 \end{bmatrix}; \quad \mathbf{A_{12\ t+1}\ H + A_{10}\ H_0} = \begin{bmatrix} 82.71271 \\ 83.18271 \\ 87.71271 \\ 88.18271 \\ 249.3705 \\ 337.5532 \end{bmatrix}$$

$$\mathbf{(NA_{11})^{-1}\ (A_{12\ t+1}\ H + A_{10}\ H_0)} = \begin{bmatrix} 0.173357 \\ 0.423658 \\ 0.446729 \\ 0.184822 \\ 0.348436 \\ 0.47165 \end{bmatrix}$$

$$\mathbf{(NA_{11})^{-1}\ A_{11}} = \begin{bmatrix} 0.539957 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.539957 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.539957 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.539957 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.539957 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.539957 \end{bmatrix}$$

$$\mathbf{I - (NA_{11})^{-1}\ A_{11}} = \begin{bmatrix} 0.460043 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.460043 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.460043 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.460043 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.460043 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.460043 \end{bmatrix}$$

$$\mathbf{(I - (NA_{11})^{-1}\ A_{11})\ Q} = \begin{bmatrix} 0.460043 \\ 0.460043 \\ 0.460043 \\ 0.460043 \\ 0.460043 \\ 0.460043 \end{bmatrix}$$

$$\mathbf{{}_{t+1}Q = (I - (NA_{11})^{-1}\ A_{11})\ Q - (NA_{11})^{-1}\ (A_{12\ t+1}\ H + A_{10}\ H_0)} = \begin{bmatrix} 0.286686 \\ 0.036386 \\ 0.013314 \\ 0.275222 \\ 0.111607 \\ -0.01161 \end{bmatrix}$$

The iterative procedure is continued further and the final solution with desired accuracy is obtained in five iterations. Iteration details are shown in the Table 1. Flow values in the fifth iteration are same up to three places after decimal. The iterative process can be continued to have more accuracy in flow values.

Table 1: *Pipe Discharges and nodal beads after different iterations*

Parameters	1	2	3	4	5
Q_1, m^3/sec	0.286686	0.166117	0.15377	0.153596	0.153596
Q_2, m^3/sec	0.036386	0.176153	0.196408	0.204748	0.205104
Q_3, m^3/sec	0.013314	0.133883	0.14623	0.146404	0.146404
Q_4, m^3/sec	0.275222	0.106857	0.051406	0.039754	0.03917
Q_5, m^3/sec	0.111607	0.08301	0.047815	0.044503	0.044273
Q_6, m^3/sec	-0.01161	0.01699	0.052185	0.055497	0.055727
H_3, m	182.7127	94.36713	92.00391	91.98018	91.98018
H_4, m	183.1827	98.14024	94.84046	94.38711	94.36166
H_5, m	432.5532	94.64127	94.01386	93.17928	93.16015

Annexure 12.6: Downtake Supply System

In the downtake supply or down feed distribution, the supply from the street main is drawn either:

1. direct into the overhead storage tank where from the supply is drawn to several floors by gravity; or
2. into a ground level storage tank where from the supply is again pumped to an overhead storage tank and then the supply is drawn by gravity.

a) High-Rise Buildings

Systems: The downtake system of water supply in high-rise buildings may be one or a combination of the following systems, viz., overhead storage system, break pressure tank system and hydro-pneumatic system.

(i) Overhead Storage System

In this system, the tanks are provided on the terrace. A manifold downtake may be taken out from the storage tank which should be laid out horizontally in a loop on the terrace to carry a designed peak load demand. The pressure in the loop at peak demand shall not become negative. Vertical downtakes, as many as necessary, may be taken from, the loop and should be linked to one downtake for a zone or four storeys at a time and designed for the peak demand it has to serve. A PRV shall be provided in the downtakes to limit the head to a maximum of 25 m in easily accessible places like ducts, cat walks, etc.

(ii) Break Pressure Tank System .

In this system, the entire building is to be conveniently divided into suitable zones of five to eight storeys each. For each such zone there shall be a break pressure tank, the capacity of which should he such that it holds 10 to 15 minutes supply, of the floor it feeds, below and shall be not than 2KL each for flushing and other domestic purposes separately. The downtake from the master overhead tank feeds into the break pressure tank.

(iii) Hydro-pneumatic System

In this system, the supply is through a hydro-pneumatic pressure vessel fitted with accessories like non-return valves and pressure relief valves. Each zone of supply should be restricted to about seven storeys or 20 m, whichever is less. The capacity of the pump should be such as to cope up with the peak demand. Normally three pumps called the lead pump, the supplementary pump, and the standby pump respectively, are provided. The last pump is preferably diesel driven to serve when there is a power failure. The hydro-pneumatic pressure vessel should be an airtight vessel, cylindrical in shape and fabricated from mild steel plates.

b) Fire Storage

Multi-storeyed buildings above 25 m height must be provided fire storages in addition to domestic needs, adequate to fight a fire at the rate of 2250 lpm, as a normal firefighting tanker cannot cope up with fires beyond an elevation of 25 m. This limit, however, varies from place to place depending upon the normal of ladder available with the local fire brigade service for extinguishing fire.

The tank capacity for fire storage may be of 100 KL, where the supply is intermittent so that it is adequate to fight a fire in the premises at the rate of 2250 lpm for about 45 minutes overflow from the firefighting tank should flow into the suction tank to maintain a continuous circulation in the static fire tank and maintain a reserve storage for firefighting purposes.

The firefighting pumps may be in the basement to have a positive suction head and designed to deliver 2250 lpm with a terminal pressure of 3 kg/cm^2 at the topmost floor to obtain from the hydrant 900 lpm discharge with a jet of about 6 m.

On the firefighting rising main, hydrant tees of 60 mm maybe provided at every landing of each staircase. A small 20 mm tapping may be provided at each landing with a wheel valve and adequate length of hose pipe for fighting small fires due to electrical short circuiting, etc.

The pump set is to be provided with a pressure vessel and automatic pressure switches which start operating when there is a pressure fall on the rising main due to the commissioning of any of the hydrant tees for fighting a fire.

To deal with cases when there is a power failure, the high-rise buildings should be provided with independent electrical circuits, one connected to the normal external power and the generator automatically come into operation in the event of power failure to supply power for all pumps including fire pumps, emergency lights, lifts, and lights in staircases and yards.

Annexure 12.7: Optimisation of Pipe Diameters

Case Study of Operational zone of 'Sajipur Old' in Ahmedabad. Operational zone of 'Sajipur Old' in Ahmedabad city (Figure 1) is illustrated. This zone is situated in North of the city and is densely populated zone. Optimisation steps as discussed in Section 12.11.4 are adopted.

Figure 1: Operational zone of 'Sajipur Old' in Ahmedabad (existing pipes in green and new pipes in blue colour)

The population of this zone is 71,506 (for ultimate design population of the year of 2050). The flow for the ultimate year (2050) with 10% losses in distribution system the pipe network in this OZ is designed to cater for the total demand of 12.46 MLD. The ESR of this zone is yet to be constructed. In view of the dense population and paucity of land, the city administration rightly decided to construct the ESR for ultimate stage. The capacity of ESR is for 8 hours, i.e., 1/3 of total demand. By this method, the requirement of capacity works out to 12.46/3, i.e., 4.15 ML.

GIS based hydraulic model is prepared. It was ensured that the boundary of the OZ was designed optimally, i.e., ESR do not get empty or get overflowing. The elevations are assigned to each node using GIS based contours and the demands are also assigned to all the nodes using the method of 'Future population density method' as described in the Annexure 2.7.

Scenario1: Earlier Design (before optimisation)

The base scenario was run with pipe diameters of the earlier consultant and the results of velocity and hf/km in the pipe table and nodal pressures in junction table are noted.

Scenario2: Optimised Design

Process: Various steps required for optimization of pipe diameters are shown in Figure 12.3 which is reproduced as Figure 2 below.

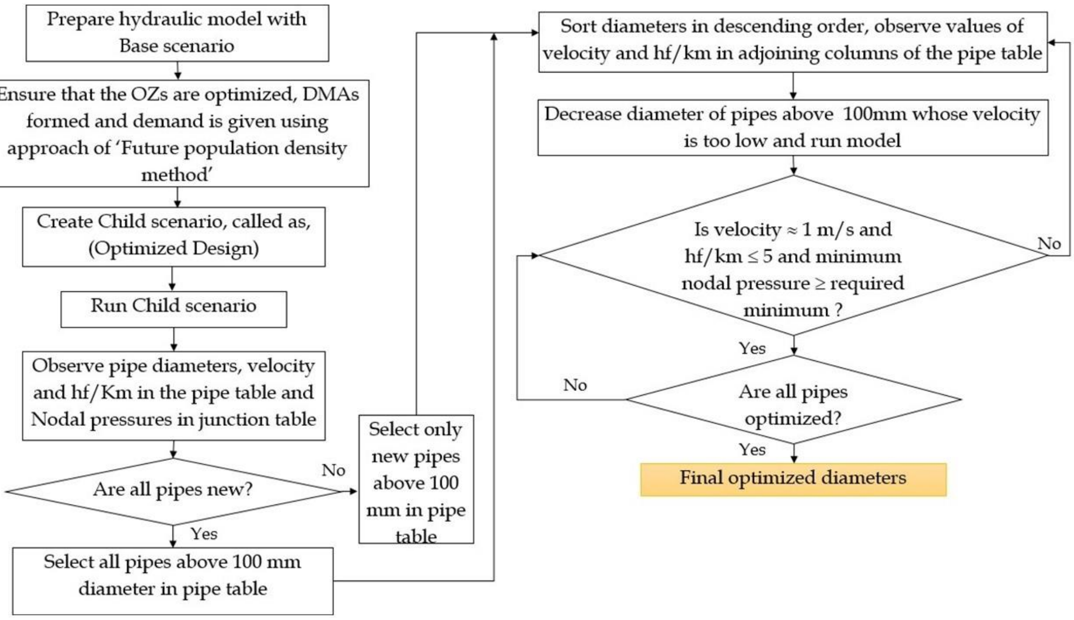


Figure 2: Iterative process of optimising diameters

Optimisation techniques from IWWA paper, "Optimisation of Distribution Pipe Network without Using Specialized Optimization Software" Journal of IWWA Oct- Dec 2020 is used here to make initial guess of pipe diameter.

These steps are followed.

a) Select pipe diameter for each pipe based on general prudence and by using capacity of pipe in MLD for velocity equal to 0.8 m/s (Column 2 of Table1 from the above paper) and assign it along with pipe label.

b) Under the base scenario, a child scenario, called as 'Optimized Design' is created. The Alternative of 'Physical' is attached to this child scenario. Alternative 'Physical' means a *data set* which can store different values of diameters other than those in Base scenario.

c) Run the Child scenario and observe pipe diameters, velocity and hf/km in the pipe table and nodal pressures in junction table.

d) Select only new pipes in pipe table and which are more than 100mm diameter. Following steps are taken for new pipes keeping existing pipes undisturbed.

e) Sort diameters in descending order, observe values of velocity and hf/km in adjoining columns of the pipe table

f) Decrease diameters of those pipes in which velocity is low and hf is also low and then run model

g) Observe the revised values of velocities in the pipe table. If velocity is less than 1 m/s and hf/km is also less than 5 m/km and minimum nodal pressure is also more than 17m (as minimum nodal pressure is 17m), then steps (e), (f) and (g) are repeated.

h) The process is repeated for all the pipes whose diameters are more than 100mm, till we get all optimized diameters.

i) The comparison of the diameters of earlier design, (ED_Diameter) and the optimised diameters (OP_ Diameter) is shown in Table 1.

j) It is experienced that one operational zone can be optimized within half hour using this technique.

Table 1: comparison of the diameters of earlier design, (ED_Diameter) and the optimised diameters (OP_ Diameter)

Scenario1: Earlier Conventional Design (before optimisation)					Scenario2: Design using "Age-old-prudence" method (after optimisation)			
Diameter (mm)	Total Length (m)	Rate (Rs/m)	Total ED_Cost (Rs)		Diameter (mm)	Total Length (m)	Rate (Rs/m)	Total OP2_Cost (Rs)
100	22362	703	15720486		100	23776	703	16714528
150	218	1,009	219962		150	6	1,009	6054
200	1415	1,322	1870630		250	213	1,734	369342
300	11	2,179	23969		300	592	2,179	1289968
400	590	3,134	1849060		400	9	3,134	28206
600	15	5,285	79275		450	15	3,575	53625
Grand Total	24611		1,97,63,580		Grand Total	24611		1,84,61,908
Length of 200mm and above= 2031m					Length of 200mm and above= 829 m			

Amount saved in one operational zone (OZ) = Rs 13 Lakhs and % saving in cost=6.6%

There are about 250 such Ozs in Ahmedabad, likely saving > Rs 25 Crores

Annexure 16.1: Case Study of the Nagpur Peri-Urban Scheme

1. INTRODUCTION

Nagpur is located at the heart of India. It is an important junction of the National Highways and Grand Trunk railway routes connecting four corners of the country. There has been unprecedented growth of the city owing to the projects of the Multi-modal International Hub and Airport in Nagpur (MIHAN). The expansion of the city has encompassed the villages on the fringes of the city boundary.

Huge tracks of agricultural lands are being converted into non-agricultural usage in these villages. It indicates large scale economic, residential, and industrial activities in the area. This area will be targeted by the people who will grab the opportunities generated by the activities of companies in different domains. Special Economic Zone (SEZ), Multi-modal International Hub Airport of Nagpur (MIHAN) also contribute a large influx of people to this area. In fact, the vigorous activities are reported from this region. It will necessitate the proper water supply and sewerage scheme. The water supply scheme, which was constructed in the past has proven to be inadequate due to exponential population growth. The villages lie just outside the limits of Nagpur Municipal Corporation (NMC). Inhabitants of the area depend upon the inadequate water supply scheme, open dug wells, and deep bore wells for their water requirement. It is, therefore, necessary to frame a long-term sustainable scheme to cater the demand of these villages.

2. EXECUTING AGENCY

The scheme was prepared by the Maharashtra Jeevan Pradhikaran (MJP). As per policy of the Government of Maharashtra (GoM), 10% of cost of the scheme, called popular contribution, was required to be borne by the peril-urban villages. But the villages refused. In such situation, MJP came forward and paid 10% popular contribution. So, the scheme has been termed as MJP's own water works.

The Nagpur Improvement Trust (NIT) was appointed as 'Special Planning Authority'. NIT makes plan of the Nagpur Metropolitan Area. They were preparing the development plan of Nagpur City and it took two years to complete. Meanwhile, the NIT had decided to frame the improvement scheme under provision of NIT Act, 1936, to cope up with the growing demand for the housing requirements of the Nagpur city. In this scheme, NIT had requested MJP to consider their nine peri-urban villages with a population of 1.65 Lakhs as of 2014, and an estimated population of 3.2 Lakhs for the year 2029. The net requirement of NIT villages was 21.7 MLD. In addition to these nine villages, MJP constructed a scheme by adding another 10 villages. Hence, MJP constructed a scheme for 19 villages, of which 10 villages of MJP are getting water by gravity from one MBR at WTP and from another MBR in the same premises of WTP; nine villages of NIT are getting water by gravity.

Here, the design of the water supply scheme of MJP of the 10 villages was considered for a demand of 13.932 MLD for the year 2029. However, both MJP's and NIT's villages were considered in discussions regarding financial sustainability. MJP is supplying water up to consumers for the 10 villages scheme through 100% consumer metering and charging them on volumetric basis with telescopic rate of tariff and is collecting revenue from consumers.

The scheme of 19 peri-urban villages has a source originating from the 'Lower Wunna' dam which is on south side of Nagpur City. The works from head works up to water treatment plant (WTP) are common for NIT and MJP. There are two MBRs at WTP site, one for the nine villages of NIT and another for the 10 villages of MJP. Details of the various components are shown in Table 1.

3. THE PROJECT

This scheme for 19 peri-urban villages is executed from the reservoir of 'Lower Wunna' dam and the works up to WTP including WTP are common. Figures 1 and 2 show the details of common works and transmission main for the 10 villages of MJP.

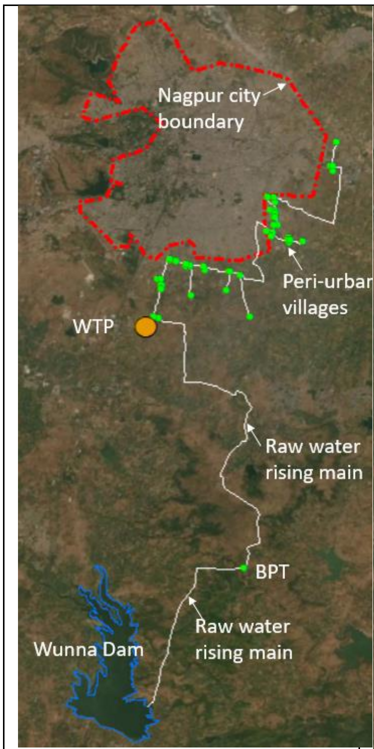


Figure 1: All elements of the existing transmission mains for 10 villages

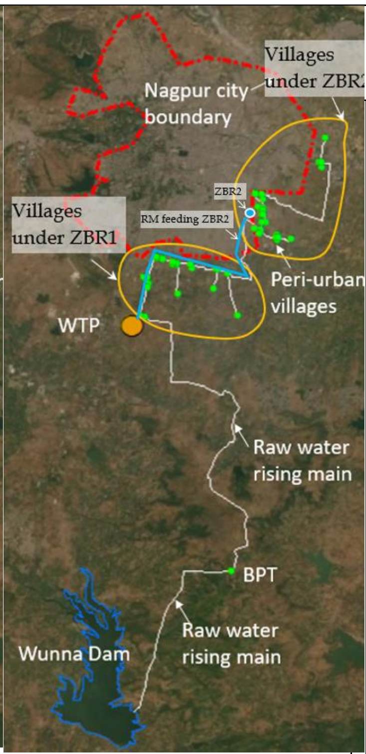


Figure 2: Suggested configuration with ZBR1 and ZBR2

Table 1: Details of various components

SN	Components	Details
1	Head Works:	Jack Well, Pump House, Approach Bridge
2	Raw Water Pumping Machinery:	5 sets, 495HP each, Q = 205 LPS, H= 118 m
3	Raw and Pure Water Rising Main	800 mm nominal dia. DI K-9.; L-13.02 Km
4	BPT	3.30 lakh lit
5	RW Gr. Main from BPT to WTP	800 mm diameter DI K-7; L-25.5Km
6	WTP for 19 villages	Capacity 47.5 mld
7	RCC MBR (Twin):	Cap 19.73 lakh lit, St. height -30 m
8	PW pumps for MJP MBR for 10 villages	5 sets, HP 170, Q=191 Lps; Head - 40 m
9	Feeder Main from MJP MBR	DI K-7, 700 mm to 100

				mm, L=55.1 km
		10	RCC MJP ESRs	27 Nos.
		11	Distribution System:	HDPE pipes 6 Kg/m^2, P.E. 100

Present Status

The scheme under discussion for 10 villages was designed for the demand of the year 2029 and was commissioned in the year 2019, since then, it is being maintained by the MJP. All consumers are metered, and revenue is collected by MJP directly. Water supply to all these 10 villages is through a total of 27 ESRs. The expanse of this peri-urban is from south to north, with WTP at the south while the last village is at the north end. Presently, all 27 ESRs are on one line and are fed by a single gravity main from MBR at WTP. Hence, the reliability of the villages is quite less as number of ESRs are more on one line.

The scheme was implemented without the concept of ZBR. For increasing reliability of villages, it is necessary to reconfigure this urban-rural scheme by locating ZBR at a strategic place. Had this been done at the time of planning the scheme, the reliability of efficiency of present scheme would have been enhanced.

The various components of the scheme are shown in Figure 3. As works are recently commissioned, retrofitting is not possible.

Survey

At the time of earlier design, a survey from source to the water treatment plant (WTP and then up to all consumers in all villages) was carried out by the MJP by using Total Station method. Fortunately, GIS coordinates, including “eastings” and “northings” of the points along roads, were available. Using this data, the GIS-based contours are generated from it.

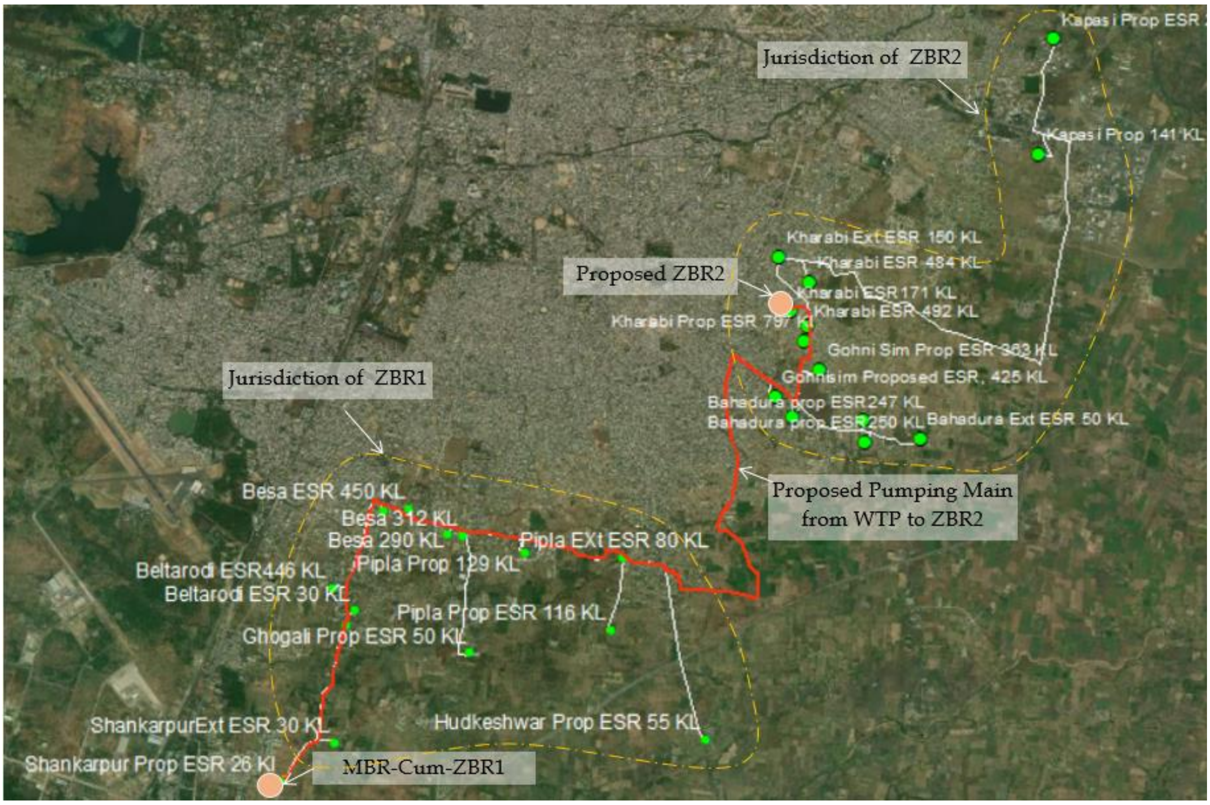


Figure 3: Rising main from the clear water pump house at WTP to ZBR1 and ZBR2

4. POPULATION FORECAST AND DEMAND ESTIMATION

Population forecast for the year 2029 and the demands of the 27 ESRs of 10 villages were computed by the MJP which was adopted for the exercise of reconfiguration, as shown in Table 2.

Table 2: ESR wise demands

SN	ZBR	Label	Cap of ESR in KL	Elevation (m)	Hydraulic Grade (m)	Demand (ML/day)
1	ZBR1	Beltarodi ESR 30 KL	30	306.937	329.648	0.072
2		Beltarodi ESR446 KL	446	319.937	328.325	1.070
3		Beltarodi Proposed 451KL	451	315.266	330.086	1.082
4		Besa 290 KL	290	317.127	321.575	0.696
5		Besa 312 KL	312	311.874	321.364	0.749
6		Besa ESR 237 KL	237	317.763	325.827	0.569
7		Besa ESR 450 KL	450	320.745	324.445	1.080
8		Ghogali Prop ESR 50 KL	50	305.928	321.145	0.120
9		Hudkeshwar Prop ESR 55 KL	55	302.282	318.199	0.132
10		Pipla Ext ESR 80 KL	80	297.012	320.206	0.192
11		Pipla Prop 129 KL	129	307.282	321.827	0.310
12		Pipla Prop ESR 116 KL	116	312.717	317.354	0.278

SN	ZBR	Label	Cap of ESR in KL	Elevation (m)	Hydraulic Grade (m)	Demand (ML/day)
13		Shankarpur Prop ESR 26 Kl	26	322.795	334.737	0.062
14		ShankarpurExt ESR 30 KL	30	312.163	333.332	0.072
15	ZBR2	Bahadura Ext ESR 50 KL	50	291.854	315.905	0.120
16		Bahadura prop ESR247 KL	247	312.34	316.385	0.593
17		Bahadura prop ESR250 KL	250	306.158	316.095	0.600
18		Gohni Sim Prop ESR 363 KL	363	311.521	315.721	0.871
19		Gohnisim Proposed ESR, 295 KL	295	305.484	317.742	0.708
20		Gohnisim Proposed ESR, 425 KL	425	305.556	315.441	1.020
21		Kapasi Prop 141 KL	141	307.282	310.481	0.338
22		Kapasi Prop ESR 230 KL	230	305.939	310.521	0.552
23		Kharabi ESR 484 KL	484	312.854	316.236	1.162
24		Kharabi ESR 492 KL	492	311.016	325.381	1.181
25		Kharabi ESR171 KL	171	308.71	323.159	0.816
26		Kharabi Ext ESR 150 KL	150	306.86	319.284	0.360
27		Kharabi Prop ESR 797 Kl	797	316.619	329.891	1.913

For analysis of the proposed peri-urban village scheme, hydraulic model is necessary. Advanced and powerful techniques of GIS mapping and hydraulic modelling have been used in the present analysis study.

5. GIS-BASED HYDRAULIC MODEL

Creation of Active Topology

Pipes: Using the tool of ‘model creator’ of the network software, the elementary hydraulic model comprising of pipes and junctions was created by using the shape files of existing and the new proposed pipes.

Levels to Junctions: The ground elevations were assigned to each node using the ‘Elevation Assigner’ method.

Demands to Junctions: Using demand assigner of the network software, demands of the year 2029 were assigned to each demand node, i.e., ESRs.

Tanks: Shape file of all 27 existing and new tanks was available. The tanks were incorporated in the model and the data of tank’s diameter, minimum water elevation, and maximum water levels, etc., was fed to the model. Thus, elementary model was prepared.

Here, “earlier design” means the design prepared by MJP, based on which the scheme was sanctioned. The scheme was designed for 15 years. So, the design was revisited and was analysed for 15 years. The pumping main was economically designed.

Demands of the year 2029 were given to the demand nodes (ESRs) and their FSL were given to the model and it was then run. It was observed that the minimum residual head was 12 m. The earlier design lacked incorporation of ZBRs. Here, it is demonstrated how to incorporate ZBRs in the transmission mains to increase the reliability of the system.

A new ZBR called ZBR2 is introduced. The location of the ZBR2 is at the highest elevation. Jurisdictions of the two ZBRs are shown in Figure 3.

6. ANALYSIS OF THE SCHEME WITH PROPOSED CONFIGURATION

Scenarios: A hydraulic model for transmission mains is created. Base scenario and child scenarios of the model are shown in Figures 4.

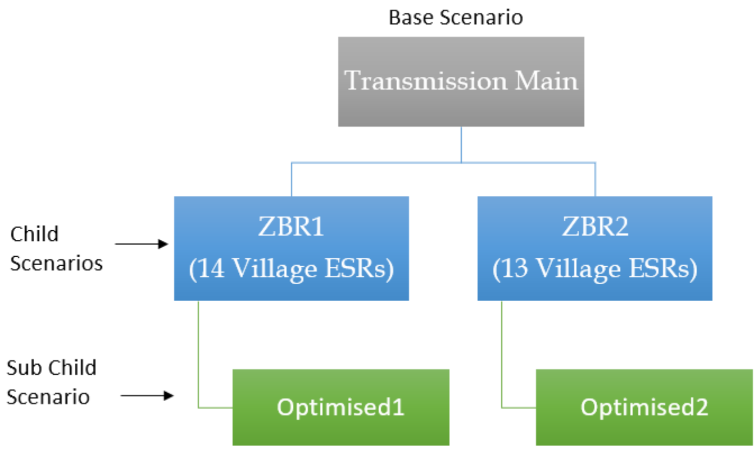


Figure 4: Model for Transmission main

7. OPTIMISATION OF PIPE DIAMETERS

Hydraulic model is run with scenarios as shown in Figure 4.

Scenario 1: Earlier Design

The base scenario was run with pipe diameters of the earlier design and the results of velocity and head loss gradient (hf/km) in the pipe table and nodal pressures in junction table are noted.

Scenario 2: Optimised Design

By following steps as shown Part A, Chapter 6 (Transmission of Water), optimised diameters are computed.

Comparison of Scenario 1 and 2

The comparison of the diameters of earlier design (ED_Diameter) and the optimised diameters (OP_Diameter) is shown in Figure 5.

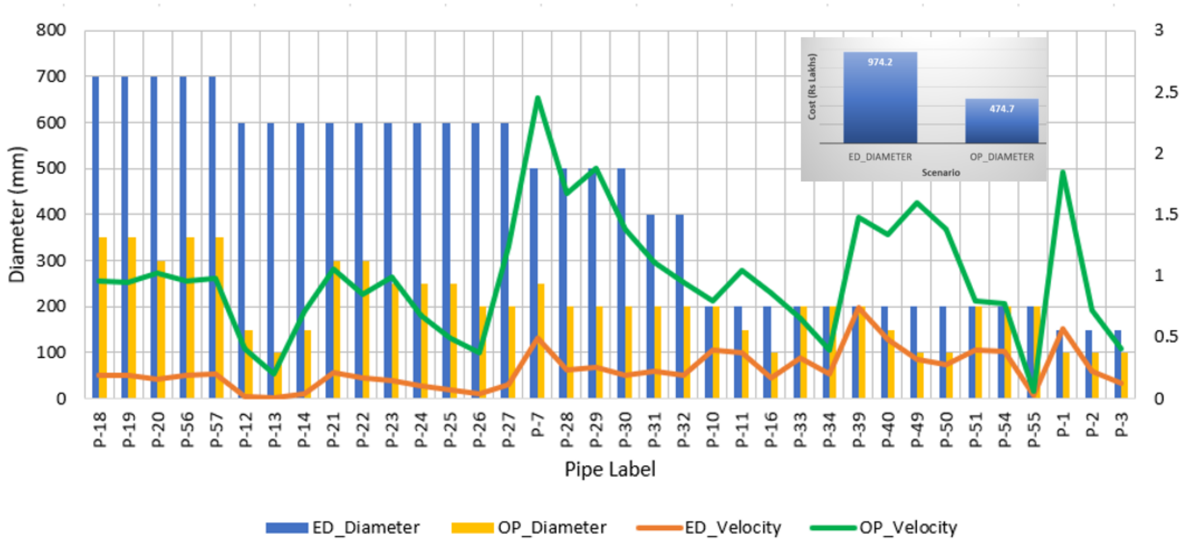


Figure 5: Comparison of diameters and velocities in Scenarios 1 and 2

The costs of the pipes computed in Scenarios 1 and 2 are also compared in Table 3. When the optimised design of Scenario 2 is considered, it is seen that there could have been a saving of Rs 114.96 Lakhs.

Table 3: Comparison of costs of pipes

Scenario1: Earlier Conventional Design				**Scenario2: Optimised Design**			
ED_Diameter (mm)	Total Length (m)	Rate (Rs/m)	Sum of Amount (Rs)	OP_Diameter (mm)	Sum of Length (Scaled) (m)	Rate (Rs/m)	Sum of Amount (Rs)2
100	5698	703	4005694	100	9806	703	6893618
150	6717	1009	6777453	150	7240	1009	7305160
200	10798	1322	14274956	200	13198	1322	17447756
250			0	250	1495	1734	2592330
300			0	300	2210	2179	4815590
350			0	350	2685	3134	8414790
400	1463	3134	4585042	400			0
500	7795	8400	32739000	500			0
600	7928	5285	41899480	600			0
700	2884	7304	21064736	700			0
Total	43283		125346361	Total	36634		47469244
				Extra rising main of 450 mm diameter	18568	3575	66380600
Grand Total	43283		12,53,46,361	Net cost	55202	3575	11,38,49,844

Explanation: Due to providing of parallel line for pumping water, the total extra length needed to be provided is (55,202-43,283) which is equal to 11,919 m.

Thus, when the scheme is designed with ZBRs, there will be a huge saving if the pipes in transmission mains are optimised.

8. EQUALISATION OF RESIDUAL HEADS

Pressure head at inlet of all the ESRs can be equalised by providing PRVs on the branch line (inlet of ESRs), solenoid altitude valve or different simple measures as discussed earlier in Chapter 6 in Part A Manual.

Here, the exercise of equalisation of residual heads at FSLs of ESRs of ZBR2 is discussed for the sake of illustration.

Computing Optimum LSL of ZBR2

Network of ZBR2 is shown in Figure 3. The model is run with active topology of pipe network of ZBR2. The hydraulic model with assumed LSL of ZBR1 at 330.0 m is run and results are noted in Column 4 and 5 of Table 4.

Table 4: Comparison of residual heads

SN	ESR Label	Demand of 2046 (ML/day)	Assumed LSL of ZBR2 at 334.795 m		LSL of ZBR2 at 329.7 m (334.79 – 5.095)	
			FSL of ESR	Residual Head (m) at FSL of ESR	FSL of ESR	Residual Head (m)
1	2	3	4	5	6	7
1	Bahadura Ext ESR 50 KL	0.12	315.91	24	315.91	3.3
2	Bahadura prop ESR247 KL	0.5928	316.39	**4**	316.39	3.1
3	Bahadura prop ESR250 KL	0.6	316.1	9.9	316.1	3.2
4	Gohni Sim Prop ESR 363 KL	0.8712	315.72	4.2	315.72	3.2
5	Gohnisim Proposed ESR, 295 KL	0.708	317.74	12.2	317.74	3.3
6	Gohnisim Proposed ESR, 425 KL	1.02	315.44	9.9	315.44	3.2
7	Kapasi Prop 141 KL	0.3384	310.48	3.2	310.48	3.2
8	Kapasi Prop ESR 230 KL	0.552	310.52	4.6	310.52	3.3
9	Kharabi ESR 484 KL	1.1616	316.24	3.4	316.24	3.4
10	Kharabi ESR 492 KL	1.1808	325.38	14.3	325.38	3.2
11	Kharabi ESR171 KL	0.816	323.16	14.4	323.16	3.3
12	Kharabi Ext ESR 150 KL	0.36	319.29	12.4	319.29	3.3
13	Kharabi Prop ESR 797 Kl	1.9128	329.93	13.3	329.93	3.1

Equalisation of Residual Pressures at FSLs of ESRs

The residual pressures of eight ESRs, as shown in Column 5 of Table 4, are quite high. It is necessary to bring down the residual pressures and make them equal or in the range of 3 to 4 m. This is done by "moving node method" as discussed in Chapter 6 (Transmission of Water).

Moving Node Method: From the main pipe of transmission main, above eight ESRs have an exclusive branch. The exercise of moving node method has been carried out for all these ESRs and the results are shown in Table 5.

Table 5: ZBR2 Network – Comparison of diameters, length, velocity, and head loss gradient in exclusive branch pipes before and after equalisation of residual heads.

<table>
<tr><th rowspan="2">SN</th><th rowspan="2">ESR Label</th><th rowspan="2">Demand of ESR (MLD)</th><th colspan="5">Before equalisation of residual head</th><th colspan="3">After equalisation of resi. head</th></tr>
<tr><th>Residual Head (m)</th><th>Diameter (mm)</th><th>Length (m)</th><th>Velocity (m/s)</th><th>Head loss Gradient (m/km)</th><th>Residual Head (m)</th><th>Diameter (mm)</th><th>Length (m)</th></tr>
<tr><td rowspan="2">1</td><td rowspan="2">Bahadura Ext ESR 50 KL</td><td rowspan="2">0.12</td><td rowspan="2">24</td><td rowspan="2">100</td><td rowspan="2">894.6</td><td rowspan="2">0.18</td><td rowspan="2">0.49</td><td rowspan="2">3.3</td><td>65</td><td>655.4</td></tr>
<tr><td>50</td><td>238.6</td></tr>
<tr><td rowspan="2">2</td><td rowspan="2">Bahadura prop ESR247 KL</td><td rowspan="2">0.59</td><td rowspan="2">4</td><td rowspan="2">150</td><td rowspan="2">191</td><td rowspan="2">0.39</td><td rowspan="2">1.31</td><td rowspan="2">3.1</td><td>150</td><td>75.3</td></tr>
<tr><td>100</td><td>115.7</td></tr>
<tr><td rowspan="2">3</td><td rowspan="2">Bahadura prop ESR250 KL</td><td rowspan="2">0.6</td><td rowspan="2">9.9</td><td rowspan="2">150</td><td rowspan="2">185.4</td><td rowspan="2">0.39</td><td rowspan="2">1.34</td><td rowspan="2">3.2</td><td>80</td><td>170.3</td></tr>
<tr><td>65</td><td>15.1</td></tr>
<tr><td rowspan="2">4</td><td rowspan="2">Gohni Sim Prop ESR 363 KL</td><td rowspan="2">0.87</td><td rowspan="2">4.2</td><td rowspan="2">100</td><td rowspan="2">323.1</td><td rowspan="2">1.28</td><td rowspan="2">19.35</td><td rowspan="2">3.2</td><td>100</td><td>296.2</td></tr>
<tr><td>80</td><td>26.9</td></tr>
<tr><td rowspan="2">5</td><td rowspan="2">Gohnisim Proposed ESR, 295 KL</td><td rowspan="2">0.708</td><td rowspan="2">12.2</td><td rowspan="2">150</td><td rowspan="2">227</td><td rowspan="2">0.46</td><td rowspan="2">1.82</td><td rowspan="2">3.3</td><td>80</td><td>223.8</td></tr>
<tr><td>65</td><td>3.2</td></tr>
<tr><td rowspan="2">6</td><td rowspan="2">Gohnisim Proposed ESR, 425 KL</td><td rowspan="2">1.02</td><td rowspan="2">9.9</td><td rowspan="2">150</td><td rowspan="2">820.3</td><td rowspan="2">0.67</td><td rowspan="2">3.6</td><td rowspan="2">3.2</td><td>150</td><td>521.1</td></tr>
<tr><td>100</td><td>299.1</td></tr>
<tr><td>7</td><td>Kapasi Prop 141 KL</td><td>0.338</td><td>3.2</td><td>100</td><td>557</td><td>0.5</td><td>3.6</td><td>3.2</td><td>100</td><td>557</td></tr>
<tr><td rowspan="2">8</td><td rowspan="2">Kapasi Prop ESR 230 KL</td><td rowspan="2">0.552</td><td rowspan="2">4.6</td><td rowspan="2">150</td><td rowspan="2">1,547</td><td rowspan="2">0.36</td><td rowspan="2">1.15</td><td rowspan="2">3.3</td><td>150</td><td>1,363</td></tr>
<tr><td>100</td><td>184</td></tr>
<tr><td>9</td><td>Kharabi ESR 484 KL</td><td>1.16</td><td>3.4</td><td>150</td><td>412</td><td>0.76</td><td>4.6</td><td>3.4</td><td>150</td><td>412</td></tr>
<tr><td rowspan="2">10</td><td rowspan="2">Kharabi ESR 492 KL</td><td rowspan="2">1.18</td><td rowspan="2">14.3</td><td rowspan="2">150</td><td rowspan="2">60.7</td><td rowspan="2">0.77</td><td rowspan="2">4.7</td><td rowspan="2">3.2</td><td>80</td><td>45.5</td></tr>
<tr><td>65</td><td>15.2</td></tr>
<tr><td rowspan="2">11</td><td rowspan="2">Kharabi ESR171 KL</td><td rowspan="2">0.816</td><td rowspan="2">14.4</td><td rowspan="2">150</td><td rowspan="2">390.6</td><td rowspan="2">0.53</td><td rowspan="2">2.4</td><td rowspan="2">3.3</td><td>100</td><td>232.2</td></tr>
<tr><td>80</td><td>158.4</td></tr>
<tr><td rowspan="2">12</td><td rowspan="2">Kharabi Ext ESR 150 KL</td><td rowspan="2">0.36</td><td rowspan="2">12.4</td><td rowspan="2">150</td><td rowspan="2">10.8</td><td rowspan="2">0.24</td><td rowspan="2">0.53</td><td rowspan="2">3.3</td><td>50</td><td>1.5</td></tr>
<tr><td>32</td><td>0.7</td></tr>
</table>

13	Kharabi Prop ESR 797 Kl	1.91	13.3	150	9.4	1.25	11.5	3.1	65	2.6
									50	10.4

From Table 5, it is seen that v (m/s) and hf (m/km) are the tools and are used prudently and effectively so that we get the optimised LSL of MBR and equalisation of residual heads at FSLs of all ESRs.

9. DESIGN OF DISTRIBUTION SYTEM OF THE PERI-URBAN VILLAGE

There are 10 villages and 27 ESRs which are supplying water to MJP's villages. The demands of the proposed 'Bahadura' ESR, with a capacity of 0.25 ML, are 0.5 and 1.67 for the years 2029 and 2044, respectively. Using online service of the GIS software, the pipe network of the distribution system is marked on GIS map including the location of the service tank as shown in Figure 6. The pipes are shown in white colour and the nodes are shown in green colour. The length of the pipelines is 5.44 km and there are 58 nodes.

Assigning Ground Elevations to Nodes: Using the facility in network software, the ground elevations are assigned to each node. There are 58 nodes. The demand for the year 2044 is 0.5 MLD. Hence, demand per node is 0.0288 MLD. Since spread of the village is small, we may not give demand using the method of future ward wise population density. Instead, a 0.0288 MLD demand is assigned to each node. Thus, the GIS-based hydraulic model is ready for run.

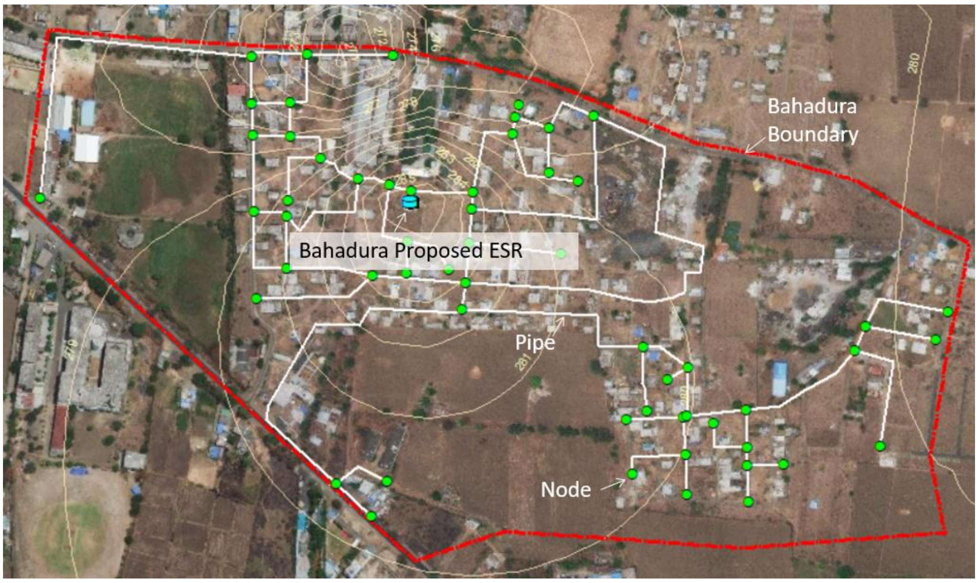


Figure 6: Distribution system of proposed 'Bahadura' ESR

Model Run: Initially, all the pipe diameters are kept minimum (110 mm HDPE pipes with inside diameter of 99.3 mm). After running the model, the results are taken. It is found that all the nodes have residual pressures of more than 22 m. This is due to the fact that the ESR is already constructed with a staging height of 22 m. When the demand for the year 2044, which is 1.67 MLD, is given to all the nodes and when the model is run, still, the nodal pressure is found more than 20 m. This means the staging height is more than required. Another striking fact is that the minimum inner diameter of pipes is 99.3 mm HDPE pipes, which is also much more than required. But as per Table 2.7 of Part A, Chapter 2, the minimum diameter considered was 100 mm. Due to higher diameter of the pipes (99.3 mm), the velocities are too less. However, there is no scope to further reduce the diameter of pipes.

10. FINANCIAL MANAGEMENT

The objective here is to determine the unit cost of production of water and then determine the tariff structure by the “Average Incremental Cost (AIC)” method. This method is discussed here for the sake of demonstration.

As the existing scheme is already executed and was designed for 15 years, the analysis is made for 15 years, i.e., from the year 2014 to 2029 as per previous JJM guidelines. However, during planning, the scheme should have been designed for 30 years.

The water supply scheme of MJP of the 10 villages is considered for an actual demand of 12.5 MLD for the year 2029 for discussions on design. However, both MJP’s and NIT’s villages are considered in discussions regarding financial sustainability. MJP is supplying water to consumers for the 10 villages scheme through 100% consumer metering and charging them on volumetric basis with telescopic rate of tariff and is collecting revenue from consumers. The scheme is commissioned in the year 2017.

Collection of Information

Information is collected from all the areas where costs are involved. This information is right from abstraction, treatment, and distribution of water. The information of forecasting of population and estimation of all demands, including domestic and non-domestic, and that of losses, are collected.

Forecasting of year wise population, the number of connections, estimation of domestic, non-domestic, educational demands of the MJP’s villages is shown in Table 6 and that of NIT’s villages is shown in Table 7.

Table 6: Forecasting of population and Estimation of all demands of MJP's villages

SN	Year	Population	Floating Population requirement (MLD)	Fire demand (MLD)	Connections				Gross requirement (including 15% losses)				Gross requirement
					Domestic	Non-Domestic	Institutional	Total	Domestic	Non-Domestic	Institutional	Total	
1	2014	44480	0.011	0.667	8896	31	30	8957	3.66	0.02	0.03	3.72	4.40
2	2015	50527	0.013	0.711	10105	35	34	10175	4.16	0.03	0.04	4.23	4.95
3	2016	56574	0.014	0.752	11315	39	38	11392	4.66	0.03	0.04	4.73	5.50
4	2017	62621	0.016	0.791	12524	44	42	12610	5.16	0.03	0.04	5.24	6.04
5	2018	68668	0.017	0.829	13734	48	46	13828	5.66	0.04	0.05	5.74	6.59
6	2019	74715	0.019	0.864	14943	52	50	15045	6.15	0.04	0.05	6.25	7.13
7	2020	80762	0.020	0.899	16152	56	54	16263	6.65	0.04	0.06	6.75	7.67
8	2021	86809	0.022	0.932	17362	61	59	17481	7.15	0.05	0.06	7.26	8.21
9	2022	92856	0.023	0.964	18571	65	63	18699	7.65	0.05	0.07	7.76	8.75
10	2023	98903	0.025	0.994	19781	69	67	19916	8.14	0.05	0.07	8.27	9.29
11	2024	104950	0.026	1.024	20990	73	71	21134	8.64	0.06	0.07	8.78	9.83

12	2025	110997	0.028	1.054	22199	77	75	22352	9.14	0.06	0.08	9.28	10.36
13	2026	117044	0.029	1.082	23409	82	79	23569	9.64	0.06	0.08	9.79	10.90
14	2027	123091	0.031	1.109	24618	86	83	24787	10.14	0.07	0.09	10.29	11.43
15	2028	129138	0.032	1.136	25828	90	87	26005	10.63	0.07	0.09	10.80	11.97
16	2029	135180	0.034	1.163	27036	94	91	27221	11.13	0.07	0.10	11.30	12.50

Table 7: Forecasting of population and Estimation of all demands of NIT's villages

SN	Year	Population	Floating Population requirement (MLD)	Fire demand (MLD)	Connections				Gross requirement (including 15% losses)				Gross requirement (without losses)	Total (MJP+NIT) Gross requirement
					Domestic	Non-Domestic	Institutional	Total	Domestic	Non-Domestic	Institutional	Total		
1	2014	40032	0.010	0.633	8006	33	20	8060	3.30	0.026	0.021	3.34	3.99	8.38
2	2015	58030	0.015	0.762	11606	39	29	11674	4.78	0.031	0.026	4.84	5.61	10.56
3	2016	76028	0.019	0.872	15206	51	38	15294	6.26	0.040	0.034	6.34	7.23	12.72
4	2017	94026	0.024	0.970	18805	63	47	18915	7.74	0.050	0.042	7.84	8.83	14.87
5	2018	112023	0.028	1.058	22405	75	56	22535	9.23	0.059	0.050	9.34	10.42	17.01
6	2019	130021	0.033	1.140	26004	87	65	26156	10.71	0.069	0.059	10.83	12.01	19.14

SN	Year	Population	Floating Population requirement (MLD)	Fire demand (MLD)	Connections				Gross requirement (including 15% losses)				Gross requirement (without losses)	Total (MJP+NIT) Gross requirement
					Domestic	Non-Domestic	Institutional	Total	Domestic	Non-Domestic	Institutional	Total		
7	2020	148019	0.037	1.217	29604	99	74	29777	12.19	0.078	0.067	12.33	13.59	21.26
8	2021	166017	0.042	1.288	33203	111	83	33397	13.67	0.088	0.075	13.83	15.16	23.38
9	2022	184015	0.046	1.357	36803	123	92	37018	15.15	0.097	0.083	15.33	16.74	25.49
10	2023	202013	0.051	1.421	40403	135	101	40638	16.64	0.107	0.091	16.83	18.31	27.60
11	2024	220011	0.055	1.483	44002	147	110	44259	18.12	0.116	0.099	18.33	19.87	29.70
12	2025	238009	0.060	1.543	47602	159	119	47879	19.60	0.126	0.107	19.83	21.44	31.80
13	2026	256006	0.064	1.600	51201	171	128	51500	21.08	0.136	0.115	21.33	23.00	33.90
14	2027	274004	0.069	1.655	54801	183	137	55121	22.57	0.145	0.123	22.83	24.56	35.99
15	2028	292002	0.073	1.709	58400	195	146	58741	24.05	0.155	0.131	24.33	26.12	38.08
16	2029	310000	0.078	1.761	62000	207	155	62362	25.53	0.164	0.140	25.83	27.67	40.17

Assumptions:

1) Per capita share of water = 135 LPCD.
2) Floating population is 1% of the population.
3) Floating population is assigned 25 LPCD.
4) Each non-domestic unit has 5 people.
5) Each Institution has 20 people.
6) Fire demand is $100\sqrt{P}$ cum/day, where P is population in thousands.
7) Average dose of alum requirement for monsoon is considered as 70 mg/L, 30 mg/L for winter season and 10 mg/L for summer. Yearly average rate works out to 36.30 mg/L, say, 40 mg/L.
8) To achieve chlorine dose of 0.2 mg/L at the far end of distribution system, chlorine dose is assumed as 2 mg/L.
9) For emergency chlorination, provision is made for 90 days requirement. Assuming 25% available chlorine in bleaching powder, for a dose of 2 mg/L, quantity of bleaching powder required is 8 mg/L.
10) Average percentage increase in energy charges per year from last 12-year data from Maharashtra State Electricity Distribution Company Ltd (MSEDCL):
    - LT 0 to 20 KW 10%
    - LT 20 to 40 KW 10%
    - LT 40 to 50 KW 9%
    - HT connection 7%
    - Average 9%
11) Charges

    (a) Fixed charges/month = No. of working sets × HP × Charges (Rs/month)

    (b) Variable charges/day = No. of working sets × KW × Charges (Rs/unit) × Pumping hours

Cost involved in O&M

Various costs involved in O&M of the water supply scheme are:

1) Establishment cost
2) Energy charges

    a. Raw Water Energy charges
        i. fixed charges
        ii. variable (as per electric consumption)

    b. Pure water Energy charges
        i. fixed charges
        ii. variable (as per electric consumption)

3) Chemical Costs: As mentioned in assumptions above
4) Raw water charges to be paid to Irrigation Department
5) Miscellaneous charges and electrification charges
6) Maintenance and Repair (M&R) charges
7) Depreciation charges

Depreciation Cost

Depreciated cost is the residual or the remaining cost of an asset after the related amount of accumulated depreciation has been deducted from it. It is the residual amount of an asset that has not yet been consumed. The formula for depreciated cost is:

$$\text{Depreciation Factor} = \frac{\text{(Cost of asset-Salvage value)}}{\text{Useful Life of an asset}} \qquad \text{(Eq. 1)}$$

Where

- Cost of the asset is purchasing price or historical cost.
- Salvage value is value of the asset remaining after its useful life.
- Useful life of the asset is the number of years for which an asset is expected to be used by the business.

There are four methods of computing depreciation. They are (i) Straight line, (ii) Double declining balance, (iii) Units of Production, and (iv) Sum of years digits.

Out of above four methods, Straight line depreciation is a common method of computation of depreciation where the value of a fixed asset is reduced gradually over its useful life. This method has been used to compute the depreciation factor in this case study. By assuming salvage value as 10% of the cost of asset,

$$\text{Depreciation Factor} = \frac{0.9}{\text{Useful Life in years}} \qquad \text{(Eq. 2)}$$

Thus, using Eq. 2 and putting the values of useful life from Column 4 of Table 8, values of depreciation factor are computed and are shown in Column 5. Multiplying the gross cost of the components of this peri-urban scheme by the depreciation factor, annual depreciation on gross cost is computed.

The M&R charges are computed by multiplying the gross cost by the established percentage of M&R and are shown in Table 8.

Table 8: Computation of depreciation and M&R

Sr. No.	**Name of work**	**Gross cost**	**Useful Life* in Years**	**Depreciation factor**	**Annual depreciation on Gross cost**	**Percentage of M&R****	**M&R charges**
1	2	3	4	5	6	7	8
1 2	Working Survey and for preparation of Detailed Project Report	27.25					
3	Head Works	634.29	67	0.01343	8.52	0.5	3.17

Sr. No.	Name of work	Gross cost	Useful Life* in Years	Depreciation factor	Annual depreciation on Gross cost	Percentage of M&R**	M&R charges
1	2	3	4	5	6	7	8
4	Raw Water Pumping Machinery	1761.78	20	0.04500	79.28	2.5	44.04
5	Raw and pure water rising main	2857.15	50	0.01800	51.43	0.5	14.29
6	BPT	55.57	67	0.01343	0.75	0.5	0.28
7	Gravity main	4786.51	50	0.01800	86.16	0.25	11.97
8	WTP	1017.63	50	0.01800	18.32	0.5	5.09
9	Pure Water Pumping Machinery	1165.91	20	0.04500	52.47	2.5	29.15
10	RCC MBR	577.21	67	0.01343	7.75	0.5	2.89
11	Feeder Main	2560.62	50	0.01800	46.09	0.25	6.40
12	Construction of RCC ESRs	924.01	67	0.01343	12.41	0.5	4.62
13	Distribution System.	2114.04	50	0.01800	38.05	0.5	10.57
14	Reform works	240.67			0.00		
15	Land acquisition	713.55			0.00		
16	Trial run	552.94			0.00		
					0.00		
		19989.13		Total	401.23		132.47

*Established in Maharashtra

**These percentages are established in Maharashtra (IWWA, 2007)

Thus, computation of annual O&M cost of both MJP's and NIT's villages are computed and are shown in Table 9.

Table 9: Computation of annual O&M cost of both MJP's and NIT's villages

Sr. No.	Year	Gross daily requirement in MLD	Yearly requirement in ml (Net)	Annual Direct Charges (Rs Lakhs)											Annual Indirect Charges (Rs Lakhs)	Annual Total O & M (Rs Lakhs)
				Establishment	Energy fixed charges (Raw)	Raw water energy charges	Energy fixed charges (Pure)	Pure water energy charges	Chemical			Raw water charges	Miscellaneous + electrification	M&R		
									Alum	Bleaching	Chlorine gas					
1	2014	8.38	3060	49.62	53.46	120.29	18.36	41.31	10.04	1.33	1.06	1.35	2.97	132.5	401.2	833
2	2015	10.56	3855	52.10	56.13	165.14	19.28	56.71	13.28	1.76	1.40	1.78	3.12	139.1	401.2	911
3	2016	12.72	4644	54.71	58.94	198.96	496.52	68.33	15.99	2.12	1.69	2.15	3.27	146.0	401.2	1450
4	2017	14.87	5428	57.44	61.89	232.55	21.25	79.87	18.69	2.47	1.98	2.51	3.44	153.4	401.2	1037
5	2018	17.01	6208	60.31	64.98	265.98	22.32	91.35	21.38	2.83	2.26	2.87	3.61	161.0	401.2	1100
6	2019	19.14	6986	63.33	68.23	299.27	23.43	102.78	24.06	3.18	2.54	3.23	3.79	169.1	401.2	1164
7	2020	21.26	7760	66.50	71.64	332.45	24.60	114.18	26.73	3.54	2.82	3.59	3.98	177.5	401.2	1229
8	2021	23.38	8533	69.82	75.22	365.55	25.83	125.54	29.39	3.89	3.11	3.94	4.18	186.4	401.2	1294
9	2022	25.49	9303	73.31	78.98	398.56	27.13	136.88	32.04	4.24	3.39	4.30	4.39	195.7	401.2	1360
10	2023	27.60	10072	76.98	82.93	431.51	28.48	148.20	34.69	4.59	3.67	4.65	4.60	205.5	401.2	1427

11	2024	29.70	10840	80.83	87.08	464.40	29.91	159.49	37.33	4.94	3.95	5.01	4.83	215.8	401.2	1495
12	2025	31.80	11607	84.87	91.43	497.24	31.40	170.77	39.97	5.29	4.22	5.36	5.08	226.6	401.2	1563
13	2026	33.90	12372	89.11	96.01	530.04	32.97	182.03	42.61	5.64	4.50	5.72	5.33	237.9	401.2	1633
14	2027	35.99	13136	93.57	100.81	562.78	34.62	193.28	45.24	5.99	4.78	6.07	5.60	249.8	401.2	1704
15	2028	38.08	13900	98.24	105.85	595.50	36.35	204.51	47.87	6.33	5.06	6.42	5.88	262.3	401.2	1776
16	2029	40.17	14663	103.16	111.14	628.16	38.17	215.73	50.50	6.68	5.34	6.77	6.17	275.4	401.2	1848

Assumptions:

(1) Establishment, Miscellaneous such as Area electrification and M&R charges are annually increased by 5%

Average Incremental Cost

This method is practised internationally (can be retrieved from UNICEF website). Incremental cost is defined as the total cost incurred due to an additional unit of product being produced. The average incremental cost (AIC) is an important tool/indicator for setting the tariff. The value of AIC gives signal to consumer to optimise the use of water.

As the demand of water increases yearly, it is important to determine the change in expense that the water utility will have to incur for increased production of water. Hence, determination of incremental cost is important. The AIC is computed using following formula,

$$\text{Average Incremental Cost (AIC)}=\frac{\text{Discounted value of total costs}}{\text{Discounted quantity of water sold}} \qquad \text{(Eq. 3)}$$

Discounted factor is defined by,

$$\text{Discounted factor} = \frac{1}{(1+r)^n} \qquad \text{(Eq. 4)}$$

Where r is the rate of interest, considering r=9%, the discounted value of all costs and discounted value of water sold at 90% billing collection efficiency are computed in Table 10.

Table 10: Discounted value of all costs and discounted value of water sold

Sr. No.	Year	Discounted factor	Annual Gross production in MLD	Annual net production in MLD (considering 15% losses)	Annual water sold and paid for (at 90% bill collection efficiency)	Discounted quantity sold yearly in MLD	Total construction cost (Rs Lakhs)	Total replacement cost (Rs Lakhs)	Total O & M Cost (Rs Lakhs)	Discounted construction cost (Rs Lakhs)	Discounted replacement cost (Rs Lakhs)	Discounted total costs (Rs Lakhs)
1	2	3	4	5	6	7	8	9	10	11	12	13
1	2014	1	3060	2601	2341	2341	0.00	0.00	833	0.00	0.00	833
2	2015	0.917	3855	3276	2949	2705	0.00	0.00	911	0.00	0.00	836
3	2016	0.842	4644	3947	3553	2990	0.00	0.00	1450	0.00	0.00	1220
4	2017	0.772	5428	4614	4153	3207	0.00	0.00	1037	0.00	0.00	801
5	2018	0.708	6208	5277	4749	3365	0.00	0.00	1100	0.00	0.00	779
6	2019	0.650	6986	5938	5344	3473	0.00	0.00	1164	0.00	0.00	757
7	2020	0.596	7760	6596	5937	3540	0.00	0.00	1229	0.00	0.00	733
8	2021	0.547	8533	7253	6527	3571	0.00	0.00	1294	0.00	0.00	708
9	2022	0.502	9303	7908	7117	3572	0.00	0.00	1360	0.00	0.00	683
10	2023	0.460	10072	8562	7705	3548	0.00	0.00	1427	0.00	0.00	657

11	2024	0.422	10840	9214	8293	3503		0.00	0.00	1495	0.00	0.00	631
12	2025	0.388	11607	9866	8879	3441		0.00	0.00	1563	0.00	0.00	606
13	2026	0.356	12372	10516	9465	3365		0.00	0.00	1633	0.00	0.00	581
14	2027	0.326	13136	11166	10049	3278		0.00	0.00	1704	0.00	0.00	556
15	2028	0.299	13900	11815	10634	3182		0.00	0.00	1776	0.00	0.00	531
16	2029	0.275	14663	12463	11217	3079		0.00	0.00	1848	0.00	0.00	507
.11	Total	9.06	142368	121013	108911	52160				21824	0.00	0.00	11419

$$\text{Average Incremental Cost (AIC)}=\frac{11419*100{,}000}{52160*1000}=\text{Rs } 22/\text{Cum} \qquad \text{(Eq. 5)}$$

AIC of Rs. 22/cum is considering the depreciation cost.

Checking value of AIC with current billing: It is shown in Table 11.

Table 11: Computation

Description of usage	Net quantity in mld	MJP's rates applicable from 1.7.2018 per unit	Col (2)*(3)
1	2	3	
Domestic 0 to 15 unit slab (85%)	4.700	17.20	80.84
Domestic 15 to 25 unit slab (85%)	0.200	26.60	5.32
Domestic 0 to 15 unit slab (85%)	0.100	36.30	3.63
Non-domestic	0.402	33.00	13.2528
Institutional	0.05	24.2	1.21
Total net	5.452		104.2528
Weighted Rate			19.12

Weighted average is just equal to the value of AIC. This means scheme is sensitive. If consumption of water varies then the scheme may go in loss.

As the rural local bodies have to sustain only O&M costs, we need not take depreciation cost, then the resulting calculations become as shown in Table 12.

$$\text{Average Incremental Cost (AIC)}=\frac{7783*100{,}000}{52160*1000}=\text{Rs } 15/\text{Cum} \qquad \text{(Eq. 6)}$$

Since without considering the depreciation cost AIC = Rs 15/cum < 22, Hence, OK

This AIC analysis has been done for 15 years, however, if the analysis is carried out for 30 years, the AIC will still reduce and is more affordable to the consumers. Therefore, this method is recommended for all the schemes including urban, urban-rural schemes, and RRWSS.

Table 12: Discounted value of all costs and discounted value of water sold

Sr. No.	Year	Discounted factor	Annual Gross production in MLD	Annual net production in MLD (considering 15% losses)	Annual water sold and paid for (at 90% bill collection efficiency)	Discounted quantity sold yearly in MLD	Total construction cost (Rs Lakhs)	Total replacement cost (Rs Lakhs)	Total O & M Cost (Rs Lakhs)	Discounted construction cost (Rs Lakhs)	Discounted replacement cost (Rs Lakhs)	Discounted total costs (Rs Lakhs)
1	2	3	4	5	6	7	8	9	10	11	12	13
1	2014	1	3060	2601	2341	2341	0.00	0.00	432	0.00	0.00	432
2	2015	0.917	3855	3276	2949	2705	0.00	0.00	510	0.00	0.00	468
3	2016	0.842	4644	3947	3553	2990	0.00	0.00	1049	0.00	0.00	883
4	2017	0.772	5428	4614	4153	3207	0.00	0.00	635	0.00	0.00	491
5	2018	0.708	6208	5277	4749	3365	0.00	0.00	699	0.00	0.00	495
6	2019	0.650	6986	5938	5344	3473	0.00	0.00	763	0.00	0.00	496
7	2020	0.596	7760	6596	5937	3540	0.00	0.00	828	0.00	0.00	493
8	2021	0.547	8533	7253	6527	3571	0.00	0.00	893	0.00	0.00	488

9	2022	0.502	9303	7908	7117	3572		0.00	0.00	959	0.00	0.00	481
10	2023	0.460	10072	8562	7705	3548		0.00	0.00	1026	0.00	0.00	472
11	2024	0.422	10840	9214	8293	3503		0.00	0.00	1094	0.00	0.00	462
12	2025	0.388	11607	9866	8879	3441		0.00	0.00	1162	0.00	0.00	450
13	2026	0.356	12372	10516	9465	3365		0.00	0.00	1232	0.00	0.00	438
14	2027	0.326	13136	11166	10049	3278		0.00	0.00	1303	0.00	0.00	425
15	2028	0.299	13900	11815	10634	3182		0.00	0.00	1374	0.00	0.00	411
16	2029	0.275	14663	12463	11217	3079		0.00	0.00	1447	0.00	0.00	397
	Total	9.06	142368	121013	108911	52160				15405	0.00	0.00	7783

Annexure 16.2: Case Study of Khambora RRWSS

1. Status before Reconfiguration of Khambora RRWSS

“Khambora” Regional Rural Water Supply Scheme comprises of 60 villages in Akola district of Maharashtra. Even though the case study is old, it is chosen for detailed

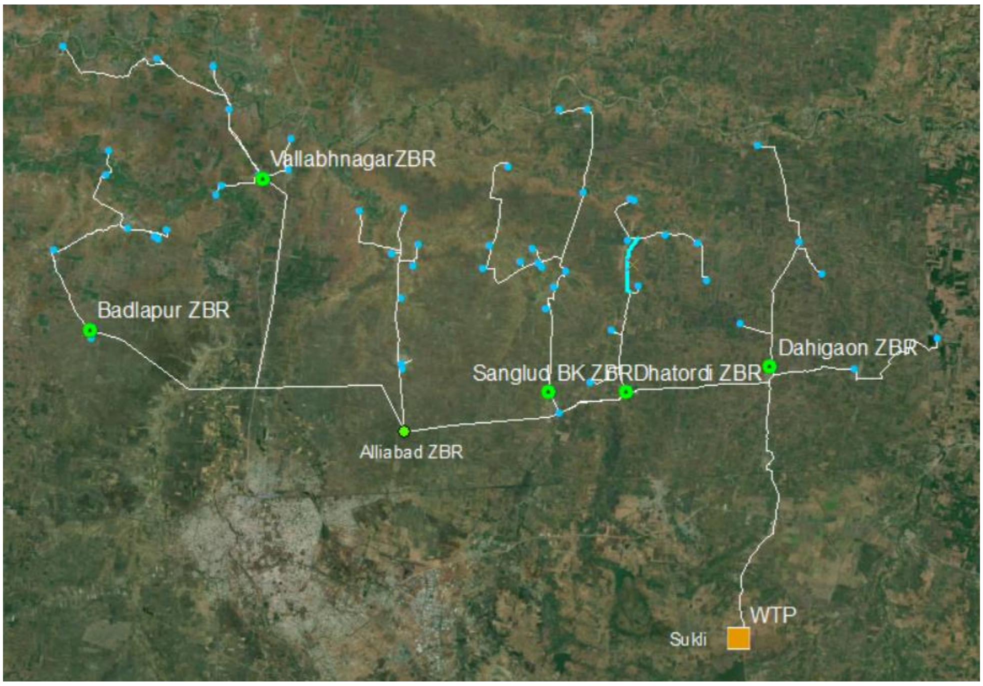


Figure 1: Configuration as per accepted tender of sanctioned scheme.
(ZBRs are shown in green colour and village ESRs in blue colour)

discussion because it covers and explains the art of creating project subareas and thereby deciding number of ZBRs, locating ZBRs at strategic places, topography, and economy in resorting to sump and pumping, location of sump, and thereby pumping in different directions. In short, it covers optimisation of diameters (capital cost) and energy cost put together along with adding very high reliability up to tail end villages. Also, it explains scale of economies of pipeline, importance of using tools of velocity (m/s) and head loss (m/km) in optimising capital and energy cost. The scheme, as shown in Figure 1, was framed in the year 1996. It was designed with only 15 years of design period, i.e., for projected population of year 2011 with the help of census population of 1991, which means the scheme was designed for 69,818 souls, 55 LPCD, and a total demand of 4.8 MLD.

The scheme was sanctioned in 1998 and work was started in 1999. Soon after that, it was decided to redesign the scheme for the projected population of 2031 (design period of 30 years beyond 2001), i.e., with 90,408 souls, 55 LPCD, and total demand of 6.29 MLD. Thus, the design demand increased from 4.8 MLD to 6.29 MLD, i.e., by 31%. Hence, total review was taken based on methodology and principles discussed earlier.

The population forecast of sanctioned scheme was reviewed and the population was again forecasted. It showed that the earlier forecast was too conservative. The span of 15 years or 30 years apparently looks to be big but is quite small as capacity of 6.29 MLD is now falling short of requirement and new scheme is now proposed for augmentation. Even though the population growth is within the forecasted population, the augmentation is needed as people are consuming more water than the norm. After deciding the scope of RRWSS, the next step is to visualise the topography of the project area.

2. Visualisation of Topography of Project Area of Khambora RRWSS (60 Villages)

In 1996, the topography of project area was visualised with help of toposheets. In the present scenario of advancement in technology, the topography, i.e., contours and slopes of the project area can be understood from "Tin-Raster Surface" shown in Figure 2.

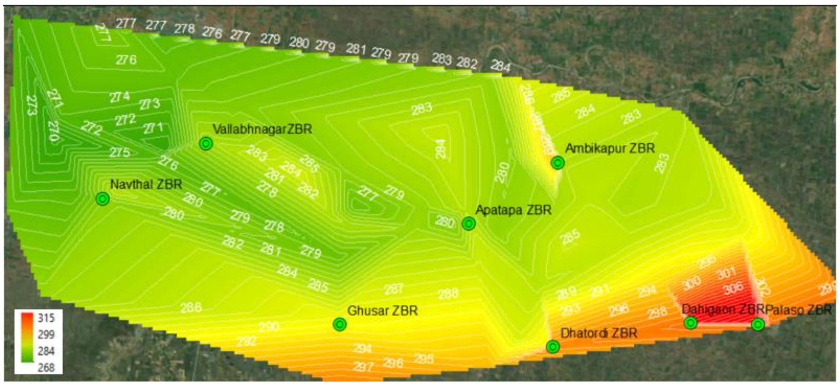


Figure 2: Tin-Raster surface help to fix the spots for ZBR

GL at WTP is 329 m, WTP is south of Dahigaon ZBR (green spot-on red area). The lowest GL of 277 m is at centre of project area and levels by and large gradually increase towards periphery of project area. Superficially, looking to the slope from WTP site to periphery of project area, one gets tempted to design a single network feeding ESRs of 60 villages from WTP sump. Theoretically, it is also possible but villages towards periphery of project area will have very less reliability. In absence of consumer metering, it is the natural tendency of villages close to WTP to draw as much as their ESR can supply and consumers in turn get as much as they can. Eventually further and further villages will starve. Hence, the accepted tender provided ZBRs but configuration of ZBR was such that on every line there were many ESRs, e.g., Sangrur ZBR, which had 12 villages on a single line. The configuration of accepted tender of sanctioned scheme is shown in Figure 1 above.

From the above Figure 1, it can be further seen that transmission main from Sangrur ZBR is not only serving 12 villages from one pipeline, but the line is also passing through the deepest GL of project area. Some of the portions of the main line from ZBRs were proposed along cart track or through fields. It is necessary that all transmission mains should be laid along all season roads for effective O&M.

In view of the above problems, it was decided to completely reconfigure the RRWSS for 60 villages. Following alternatives were hydraulically and financially analysed. For all alternatives same rates for pipe (Rs/m) were adopted. Criteria of pipelines above 200 mm diameter in DI and pipelines of 200 mm diameter and below in PVC of minimum class of 6 kg/cm^2 was kept same for all alternatives. Distribution water to villages at the locations of ZBRs were served directly from ZBR. In few cases where two villages were adjoining with each other, they were served from single ESR. In short, basic parameters were same for all alternatives. The comparison is kept limited for discussion on capital cost and energy cost of transmission main from WTP to ZBRs, ZBRs to ESRs, MBR, additional sump and pure water pumps at WTP and at additional sump. Cost towards other components like increase in diameter of raw water rising main, increase in capacity of WTP, increase in HP of raw water pump, increase in capacity of ESRs of villages are not considered in the discussion. There is no change in head work as explained below.

The source of the scheme is a pick-up weir built across the 'Katepurna' river. There is a dam on upstream side of this pick-up weir which is called as 'Katepurna' dam. Water from the pick-up weir, is pumped to the water treatment plant (WTP). Hence, the source is reliable and dependable. The head works of 8 m diameter and 10.5 m deep with overhead pump house was kept unchanged. The WTP is located at high elevation (329 m) near village 'Sukli' which is in between the project area and the source. The basic criteria of choosing a reliable source with 95% dependability at minimum distance from project area and taking advantage of high elevation for WTP is by choosing location near to on way village is fulfilled, so that raw water rising main is along all season road and WTP is not situated at a remote place.

3. Optimisation of Capital Cost Plus Energy Cost by Hydraulic Analysis of 3 Alternatives

The analysis of the three alternatives was carried out for the three alternatives as below:

Alternative No. 1: Without any change in configuration of the sanctioned scheme (Figure 3a), the scope is increased to year 2031, i.e., the water is to be gravitated from WTP sump to ZBRs and further to ESRs. The cost of pipes from WTP to ZBRs works out to Rs.16.87 Crores. The velocities in the pipelines up to ZBRs are found to be in the range of 0.23 to 0.57 m/s.

In spite of difference in elevation of 14 to 37 m. in lowest supply level (LSL) of WTP sump and FSL of ZBRs, the velocities are very low and in turn, cost of pipes is very high. Similar is the case for pipeline from ZBRs to ESRs and the cost of pipes amounts to Rs. 6.91 Crores. Residual head of 5 m is considered. The total cost of pipes from WTP to ZBRs to ESRs is Rs. 23.8 Crores.

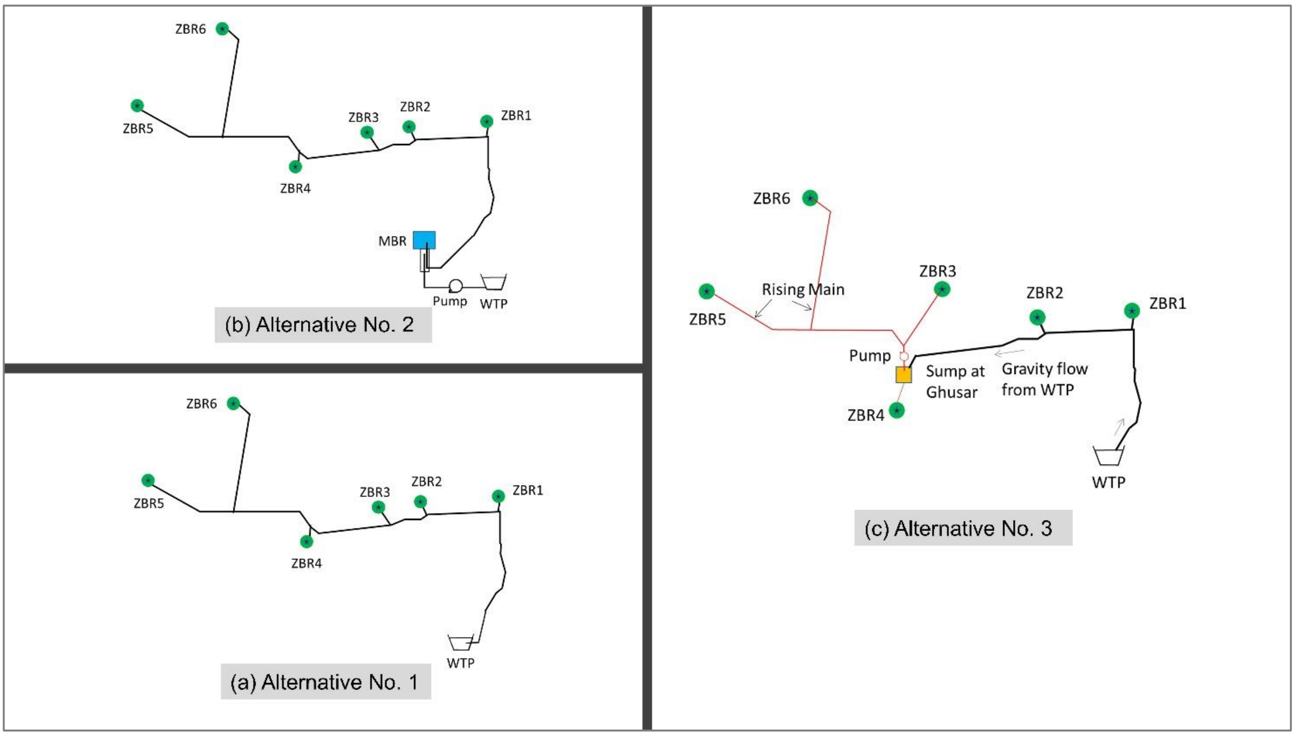


Figure 3: Alternatives

Alternative No. 2: For improving velocities in pipes from WTP sump to ZBRs, the elevated MBR with pure water pumps is needed/proposed at WTP (Figure 3b), which reduces the cost of the pipes from WTP to ZBRs to Rs. 14.02 Crores. With detailed calculations, it was seen that capital cost of MBR plus capital cost of pumps and capitalised cost of energy put together worked out to Rs. 79.30 lacs. By addition of MBR, the velocities in this alternative for pipes from MBR to ZBRs are increased to the range of 0.42 m/s to 0.71 m/s. The heights of ZBRs could also be increased to the extent of optimising cost and the residual head of 5 m could be maintained. Due to increased height of ZBRs, the cost of pipes from ZBRs to ESRs reduces to Rs.6.09 Crores compared to Rs.6.91 Crores of alternative No. I. The total cost of pipes from WTP to ZBRs to ESRs reduces to Rs.20.12 Crores compared to Rs.23.8 Crores of alternative No. 1. After considering the capital cost and capitalised cost of Rs.79.30 lacs as above, the savings on alternative No. 1 is Rs. (23.8-20.9=) 2.9 Crores, i.e., 12.1%.

Alternative No. 3: The land in the project area is a rich agricultural land and villagers did not allow to develop roads. Hence, few of the villages were on cart tracks which are not even Jeepable with four-wheel drive-in monsoon period of four months. In the configuration of sanctioned scheme, substantial length of leading mains up to ZBRs was either along such cart tracks or cross country. Additionally, leading mains from WTP to ZBRs were serving 6 to 12 villages from outlets from ZBRs. Few of these leading mains in their initial stretches were along cart tracks unreachable in monsoon and hence reliability of further dependent villages was much reduced.

To overcome all these problems, for achieving four villages on each outlet from ZBRs, the configuration was changed by changing the places of the ZBRs to strategic locations, i.e., near to the groups of villages. With above principles, six groups of villages, location of ZBRs for each group was finalised as shown in (Figure 3c).

Figure 4: Configuration of Scheme as per actual execution (total length of pipelines from WTP to ZBRs to ESRs is 175.4 kms).

(ZBRs are shown in green colour and village ESRs in blue colour)

LSLs and FSLs of ZBRs were finalised from the FSLs of ESRs in respective groups. That showed that the ZBRs at Apatapa, Vallabhnagar, Navthal and Ghusar need pumping so that reasonable velocity can be achieved in the principal transmission main for the flow of 2031 with MBR at WTP and that MBR with required pumps at WTP can be postponed till 2016. From Figure 4, it can be visualised that location of Ghusar pump near Ghusar ZBR is fairly equidistant from the other three ZBRs. Separate rising mains were feasible for Apatapa and Ghusar ZBRs and were designed with the principles of economical diameter. For Vallabhnagar ZBR and Navthal ZBR, branched rising main was necessary for avoiding parallel line. Aliabad, located on southwest side, needed local pumping. Three villages on east side of Dahigaon ZBR are situated at high elevation and, hence, their feeding directly from principal transmission main as per original proposal was retained in immediate stage.

In ultimate stage along with MBR at WTP, ZBR for these villages becomes feasible and can be added. Also, separate ESR for the villages at the places of ZBRs can also be added. As shown in Figure 4, a pumping station at a strategic location, i.e., at village Ghusar is introduced, which increases the velocities in the gravity mains from WTP and increased velocity were in the range of 0.67 to 1.12 m/s.

The velocities from ZBRs to ESRs are also increased because of increased height of ZBRs and maintaining four villages on each outlet and, hence, from total length of 175.5 km, a length of 109 kms was needed to be of 200 mm and less diameter and has been provided in PVC while a minor length of 2.0 km is needed to be of 250 mm DI. For nalla crossing, a length of 2.5 km is considered in DI for lower diameters.

Comparison of total costs, i.e., capital cost of pipes, sump and pumps and capitalised cost energy put together is shown in Table 1.

Table 1: Comparison of total costs

Alternative No. 1 (Flow totally by gravity from WTP sump)	**Alternative No. 2** (Flow by gravity from MBR at WTP)	**Alternative No. 3** (Flow by gravity from WTP sump and pumping at Ghusar)
ZBRs height is less	ZBRs height is more	ZBRs height is more
Design Year 2031	Design Year 2031	Design Year 2031
23.8	20.9	16.9

The above exercise was carried out for the pumping hours of 16 hours. However, in other but similar cases, if the pumping hours is 20, and if all the ZBRs get water by gravity from the MBR, then pumping need not be made as discussed in above case study.

4. Conclusions

1) A proper configuration of large RRWSS should be done by locating ZBRs at centres of groups of villages in such a way to have around 3–4 villages on each outlet of ZBR which increases reliability of tail end villages.
2) Maintaining adequate velocities in pipelines reduces the capital cost substantially.
3) Appropriate location of needed sump and pump house apportions flow to ZBRs in different directions.

4) The total cost of alternatives should be compared, i.e., by including the capital cost of sump, pumps, and capitalised cost of energy. For computing energy cost, unit rates of electricity without subsidy need to be considered. Comparison of total costs is shown in Table 1.
5) Velocity range for Alternative 2 is better than Alternative 1, and Alternative 3 has achieved the best range of velocities.
6) Alternative 3 is found most economical despite reducing maximum number of villages in series on a single line from 12 to 4.

BIBLIOGRAPHY

BIBLIOGRAPHY

[CHAPTER RELEVANT BIBLIOGRAPHY]

Chapter 1

1. Census, "Analysis of Census 2011 Data"
2. Census, "Household Amenities and Assets," 2011
3. CPHEEO Manual (1999). Manual on Water Supply and Treatment. Ministry of Urban Development, Government of India, New Delhi.
4. Environmental Hygiene Committee Report, 1949, Available at http:// www.worldcat.org/ title/ report-of-the-environmental-hygiene-committee-october-1949
5. NITI Aayog's Composite Water Management Index (CMWI), 2019

Chapter 2

1. Abdul Shaban and R N Sharma , "Water Consumption Patterns in Domestic Households in Major Cities," Economic and Political Weekly June 9, 2007
2. Alcubilla, R. G., Lund, J. R. (2006). "Derived willingness to pay for household water use with price and probabilistic supply", Journal of Water Resources Planning and Management, 132(6), 424-434.
3. DMA Management Manual, published by IWA, 2006
4. Hassan Tolba Aboelnga et al, "Urban Water Security: Definition and Assessment Framework" MDPI, Vol. 8, Issue 4, Sept 2021
5. Haestad Methods, 2003, available at https://www.researchgate.net/publication/ 221936180_Advanced_Water_Distribution_Modeling_and_Management
6. Oxford Dictionary, "Optimization" available at www. lexico.com/en/definition/optimization
7. PAS-SLB data from www.pas.org.in covering 900 cities in 5 States
8. PAS, "Road Map of 24x7 projects in Maharashtra," A report of CEPT University Ahmedabad, 2013
9. PPIAF Publication (2006). "Approaches to Private Participation in Water Sector - A Toolkit", The World Bank -PPIAF Publication.
10. Vali Ghorbanian, "Pressure Standards in Water Distribution Systems," J. Water Resource Planning and Management, 2016, 142(10).
11. WHO Manual, "Leakage Management and Control - A Best Practice Training Manual," 2001.

Chapter 4

1. Schroder, J. J., Scholefield, D., Cabral, F. and Hafman, G. (2004). "The effects of nutrient losses from agriculture on ground and surface water quality: the position of science in developing indicators for regulation", Environmental Science and Policy, 7(1), 15-24.
2. Kumar, Mohan MS and Manohar, Usha and Pallavi, MRM and Anjana, GR (2013) "Urban water supply and management". In: Journal of the Indian Institute of Science, 93 (2). pp. 295-317.

Chapter 5

1. Igor J. Karassik (2008), "Pump Handbook edited by Karassik"
2. Bureau of Energy Efficiency, Pumps and Pumping System, https://www.beeindia.gov.in/sites/default/files/3Ch6.pdf

Chapter 6

1. Bhave, P. R. (2003). *Optimal Design of Water Distribution Networks.,* Narosa Publishing House, New Delhi.
2. Deb, A. K. (1978). "Optimization in design of pumping system". *J. Environmental Engineering Division*, ASCE, 104(1), 127-136.
3. DIPRA, "Maximum Water Velocity in Cement Mortar Lined Pipelines," retrieved from- https://www.wsscwater.com/files/live/sites/wssc/files/pipeline%20design/2017WSSCPipeline Design ManualMay-2017.pdf
4. *Ingle, R. N., "Development of Guidelines for sizing Break Pressure Tank," Afro-Asian* Conference II.
5. IWWA, Pune, "Technical Data Book," 4th Edition, 2007
6. Osborne, J. M., and James, L. D. (1973). "Marginal economic applied to pipeline design". Transportation Engineering J., ASCE, 99(TE3), 637-653.
7. Dr. S. Sundaramoorthy, Dr. A Mani and A.L. Radhakrishnan, CMWSSB, Report on "Ring Main System of Chennai", 2023
8. Gupta, R., Mankar, S. and Bhave, P. R. (2008). "Optimal design of pumped water transmission systems", 40th Annual Convention of IWWA, Indore, 97-100.
9. Kadu, M. S., Gupta, R., and Bhave, P. R. (2008). Optimal design of water networks using a modified genetic algorithm with reduction in search space. J. Water Resour. Plann. Manage. 134(2), 147–160.

Chapter 7

1. BIS, "IS 10500:2012 - DRINKING WATER – SPECIFICATION"
2. Ministry of Jal Shakti, GoI, "Drinking Water Quality Monitoring & Surveillance Framework", 2021

Chapter 8

1. Anbazhagi, M., Loganathan, D., Tamilselvan, S., Jayabalou, R., Kamatchiammal, S., & Kumar, R. (2007). Cryptosporidium oocysts in drinking water supply of Chennai City, southern India. CLEAN-Soil, Air, Water, 35(2), 167-171
2. Daniels, M. E., Shrivastava, A., Smith, W. A., Sahu, P., Odagiri, M., Misra, P. R., Panigrahi, P., Suar, M., Clasen, T., & Jenkins, M. W. (2015). Cryptosporidium and Giardia in Humans, Domestic Animals, and Village Water Sources in Rural India. The American journal of tropical medicine and hygiene, 93(3), 596-600. https://doi.org/10.4269/ajtmh.15-0111
3. LeChevallier, M.W., W.C. Becker, P. Schorr, and R.G. Lee. 1992. "Evaluating the Performance of Biologically Active Rapid Filters." J. AWWA. 84(4):136-146.
4. Satoshi Fukuzaki (2006), "Mechanism of Actions of Sodium Hypochlorite in Cleaning and Disinfection Processes", retrieved from https://www.jstage.jst.go.jp/article/bio1996/11/4/11_4_147/_pdf/-char/en

5. USEPA, 2010, LONG TERM 2 ENHANCED SURFACE WATER TREATMENT RULE TOOLBOX GUIDANCE MANUAL
6. USEPA, 1999, Alternative Disinfectants and Oxidants Guidance Manual
7. Utaaker, K. S., Joshi, H., Kumar, A., Chaudhary, S., & Robertson, L. J. (2019). Occurrence of Cryptosporidium and Giardia in potable water sources in Chandigarh, Northern India. Journal of Water Supply: Research and Technology-Aqua, 68(6), 483-494.
8. World Health Organisation (WHO), 2022, Guidelines for Drinking Water Quality

Chapter 9

1. Agarwal, A., Ng, W.J., Liu, Y. (2011). “Principle and Applications of Microbubble and Nanobubble Technology for Water Treatment”, Chemosphere, 84 (9), 1175–80.
2. Baljinder Singh, Natun Shukla, ChanHyun Chao, et al Korean Society of Environment Risk Assessment and Health Science, , Effect of application of Micro and nanobubbles in water purification, Toxicology and Environmental Health Science, 2021
3. Batagoda, J.H., Aluthgun, H.D., Meegoda, J.N., Nano-ozone bubbles for drinking water treatment, Journal of Environmental Engineering and Science 2019 14:2, 57-66.
4. C. Rameshkumar, G. Senthilkumar and R. Subalakshmi, Stahyabama Institute of Science and Technology, University of Madras, Chennai, Purification of Tap Water to Drinking Water nanobubbles technolog, Desalination and Water Treatment., 2021
5. Rook, J.J. (1974). “Formation of Haloforms during Chlorination of Natural Water.” Water Treatment and Examination, 23(2): 234-243
6. Tatek, T., Bui, T.T., Han, M., Kim, T.I., Park, H. (2017), “Micro and Nanobubble Technologies as a New Horizon for Water-Treatment Techniques: A Review”, Advances in Colloid and Interface Science, 246, 40–51.
7. USEPA Report, Wastewater Technology Fact Sheet Ozone Disinfection, EPA 832-F-99-063, 1999.
8. Batagoda JW, Hewage SDA and Meegoda JN Nano-ozone bubbles for drinking water treatment. Journal of Environmental Engineering and Science, https://doi.org/10.1680/jenes.18.00015

Chapter 12

1. Abdy Sayyed, M. A. H., Gupta, R., and Tanyimboh, T. T. (2015). Noniterative application of EPANET for pressure dependent modelling of water distribution systems. Water Resour. Manage., 29(9), 3227–3242.
2. Abdy Sayyed, M. A. H., Gupta, R., and Tanyimboh, T. T. (2019). Combined flow and pressure deficit-based penalty in GA for optimal design of water distribution network, SH Journal of Hydraulic Engineering, DOI: 10.1080/09715010.2019.1604180.
3. Alcubilla, R. G., Lund, J. R. (2006). “Derived willingness to pay for household water use with price and probabilistic supply”, Journal of Water Resources Planning and Management, 132(6), 424-434.
4. Bhave, P.R. (1978). “Non computer optimization of single source networks.” J. Env. Eng., 104(4), 799-814

5. Bhave, P. R. (1981). Node flow analysis of water distribution systems. J Transp Eng 107(4):457-467.
6. Bhave, P. R., Gupta, R, (2006). Analysis of Water Distribution Networks. Narosa Publishing House Pvt. Ltd. New Delhi, India.
7. Chandapillai, J. (1991). Realistic simulation of water distribution systems. J Transp Eng 117(2):258-263.
8. DMA Management Manual, published by IWA, 2006
9. ESRI, "Create and Share Maps, Analytics, and Data," available at desktop.arcgis.com
10. Gupta, R., Bhave, P. R. (1996). Comparison of methods for predicting deficient network performance. J Water Resour Plann Manage 122(3):214-217.
11. Kalpana Bhole, Ph.D. Thesis, VNIT, Nagpur, 2014
12. Majumdar, J. (1990), “Finance and economics”, http:// www.lboro.ac.uk/departments/cv/ wedc/ publications /w2/paper5.pdf, (Feb. 18, 2009).
13. M. R. Mulay, Bhole, K.S., (Nov 2020), Dahasahasra S.V., "Test for Achieving 24x7 Water Supply," International Journal for Research in Applied Science & Engineering Technology (IJRASET), ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor:
14. NEERI Report (1991). Training program on preventive maintenance of water distribution systems, held at Nagpur under sponsorship of WHO and CPHEEO, Govt. of India,
15. Network Without Using Specialized Optimization Software,” Journal of IWWA, October-December, 2020, 247-257
16. R S Mckenzie & W. Wegelin, “Implementation of Pressure Management in Municipal Water Supply Systems,” Available at https://www.semanticscholar.org
17. Sandra Lach Arlinghaus, “Bisectors, Buffers, and Base Maps,” The Web Environment at U-M, Available at http://www-personal.umich.edu.
18. Wagner, J. M., Shamir, U., Marks, D. H. (1988). Water distribution reliability: simulation method. J Water Resource Planning and Management 114(3):276-294.

Chapter 16

1. Domkondwar, M.V., “Configuration of Regional Water Supply Scheme for Effective Operation and Maintenance,” IWWA, Oct-Dec, 2000
2. Ministry of Jal Shakti, GoI, “Handbook for Sarpanch & Gram Panchayat Functionaries Capacity Building of Gram Panchayats,”2019, available at swachhbharatmission.gov.in/sbmcms/writereaddata/Portal/Images/pdf/Website-%20Sujal%20and%20Swachh%20Gaon_Handbook_ENGLISH%2009092019.pdf
3. National watermission, “Categorization of Blocks/ Mandals/ Talukas in India (March, 2011) available at http://www.nationalwatermission.gov.in
4. Report of the Working Group of Twelfth Five Year Plan on, "Rural Domestic Water and Sanitation” Ministry of Drinking Water and Sanitation. Government of India, September 2011
5. WRIS, “Map of INDIA”

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