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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 lay-out 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 meter above the sewer lines.
d) Joints should be water-tight 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.

Intermittent Water Supply System

The intermittent system has major disadvantages of contamination, high NRW, inequitable distribution of water supply and high coping cost. The main causes of intermittent supply are as follows:

- Improper planning, design, layout of network and poor O&M
- Unmetered supply
- Improper selection of pipe material
- Pipes are too old and not replaced
- Non-availability of adequate quantity of water at source leading to rationing of water supply to different areas by adopting practice of zoning
- Unexpected or unbalanced growth during design period
- Improper peak factor
- Heavy leakage losses leading to high NRW
- Service reservoirs not having adequate staging height to meet the residual pressure at ferrule
- Continuation of Water Distribution System (WDS) beyond its design life

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 24x7 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.

Continuous Water Supply System

24x7 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.

Advantages of conversion to Drink from Tap (24x7) water supply system are as follows:

- Improved health
- Improved water service quality
- Operation under continuous supply to reduce leakage and save water

- Improved revenue
- Improved living conditions
- Women and children get benefitted.
- Poverty reduction
- Improved conditions for inward investment

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.

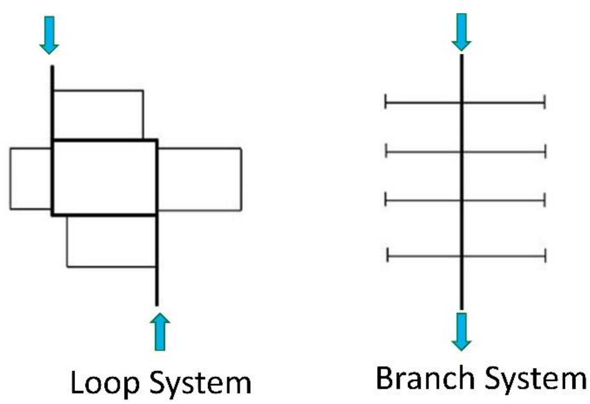

Figure 12.1: Pattern of Distribution System

12.2.3 Condition Assessment and Integration of Existing Network

Details are mentioned in Chapter 2 “Planning, Design 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, which is usually a combination of branched and looped network is provided.

Loops in the network helps 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 & 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 is the division of project area into smooth manageable areas. Operational Zone in the distribution system ensures equalization 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 and (e) facility for isolating for assessment of waste and leak detection (f) type of activity (g) physical barrier like expressway, canal river, railway track. The operational zone 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, IDW (Inverse Distance Weighted) 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 operational zone in the distribution system. If there is an elevation difference more than 20-25m then the operational zones 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 5m.

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 operational zones/ DMA is essential in this case. Operational Zones (area served by a service reservoir) must be separated and pressure reducing valves 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.

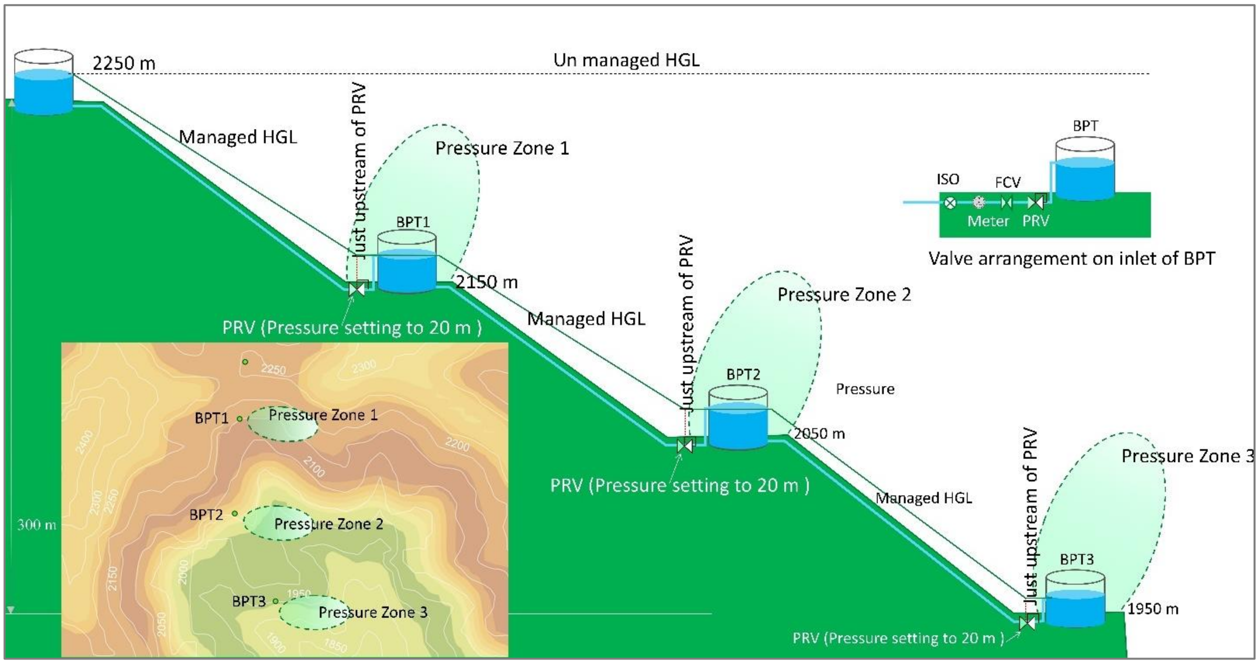

Figure 12.2: Pressure zones in hilly city

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 zone 1, 2 and 3 respectively. Before each of BPT a Pressure Reducing Valve (PRV) is installed which is set to limit the zone pressure to 21m. 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 21m. Direct-acting PRVs can be used to limit pressures up to 21m.

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 (FSL) of water in Elevated Service Reservoir (ESR) shall approximately be equal to highest ground level in operational zone + required minimum residual head + frictional losses.

A suitable combination of pipe sizes and staging height has to be determined for optimization of the system. With concept of decentralised planning, head loss gradient up to 5m/km can be achieved. Presently, 10 to 12 storey buildings are quite common and construction technology is quite modernized, hence the ESR could be provided with high staging height. However, the maximum staging height of Elevated Service Reservoir (ESR) should be properly designed.

Equitable distribution of water with designated pressures in 24x7 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. Equalization 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)/Pressure Reducing Valve (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 then FCV in sequence in direction of flow are provided. The FCV is set such that the inflow to the tank would be as per demand of the operational zone (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 24x7 continuous water supply system, online boosting with Variable Frequency Drive (VFD) pump at following locations in the Operational Zone (OZ) of existing ESRs can be provided for achieving required minimum nodal pressures for continuous water supply system:

- On-line boosting at the outlet of tank for entire OZ having residual nodal pressures less than 21 m or 15 m (as the case may be).
- On-line boosting for the branch line to area having residual nodal pressures less than 21 m or 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 meters, 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 operational zone/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, 150mm 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 ensure suitable residual pressure to the distribution system and minimize 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 shall be as per Table 2.7 of the Chapter 2. 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.

In order to speed up the Construction activity & to deliver the Water to the Consumer on priority, designer can opt for speedy construction methods Like Precast RCC Staging with Prefabricated Metal Container.

<u>Below mentioned are the specifications which needs to be followed for Precast RCC Staging:</u>

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 de-molding shall be minimum 15Mpa.
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 & Coupler is:
   - Compressive strength at 6hrs is >15N/mm2
   - Compressive strength at 24hrs is >30N/mm2
   - Compressive strength at 28days is >65N/mm2
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)
    - IS: 456 -2000 – Code of practice for plain and reinforced concrete
    - IS: 875 -2015 - Code of practice for Design Loads for Building and Structures.(Part-1 to 5)
    - IS: 1893-2016 - Criteria for Earthquake Resistant design of structures.
        - Part-1 General Provisions of buildings

- IS: 1893-2014 - Criteria for Earthquake Resistant design of structures.
  - Part-2 Liquid Retaining Tanks
- SP: 34 - Handbook on Concrete Reinforcement and Detailing.
- 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 & 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 300MPa.
4) Base material of Roof sheet shall be of Steel with minimum yield strength of 550MPa.
5) Composition of coating on the base material i.e., Zn, Al & Si shall be 43.5%,55% & 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 0.8mm.
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 with the base slab with Bolts & Stiffener arrangement to hold the water inside the Container, Food grade reinforced PVC liner of minimum 890GSM thickness shall be used.
10) Serviceability of Liner shall be for a range of external Temperatures from -5°C to +70°C

12.4.4 Inlets and Outlets

The draw pipe should be placed 15 centimetres above the floor and is usually provided with a strainer of perforated cast iron. The reservoirs filled by gravity are provided with 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. 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 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 nonlinear, and for the pressurised flow in pipe either Darcy-Weisbech or Hazen-Williams equation (Chapter 6) can be used. The pipe head loss relationship in a general form can be written as

$$h = R\, Q^n \qquad (12.1)$$

where, h is head loss in pipe, Q is discharge in pipe, and R is called resistance of the pipe, n is exponent of discharge in Darcy-Wesbech / 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 & 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

$$\sum_{x \epsilon j} Q_x - q_j = 0 \qquad (12.2)$$

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:

$$\sum_{x \epsilon j} Q_x - q_j \pm \frac{dS}{dt} = 0 \qquad (12.3)$$

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.

$$\sum_{x \epsilon l} h_x = 0 \qquad (12.4)$$

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 Equations. (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 (Bhave and Gupta 2006). The commonly used methods are the Hardy Cross Method, Newton-Raphson Method, Linear Theory Method and the Gradient Method. There are few other methods like analysis using electrical analysers, analysis using unsteady state behaviour during start up, analysis through optimization and analysis using perturbation method. These methods are not common methods; however, these have been suggested in the literature.

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:

$$\Delta Q = \frac{-\sum h}{n.\sum {}^{h}/_{Q}} \qquad (12.5)$$

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. 12.5, 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:

$$\Delta H = \frac{\sum Q}{\sum {}^{Q}/_{n\,h}} \qquad (12.6)$$

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. (12.6).
(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;

$$\Delta p = \frac{f(p)}{f'(p)} \qquad (12.7)$$

The nature of this result can be recognized 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) in Eq. (12.8).

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 ΔQ_i, 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 program terminates at the allowable head tolerance or when iterations exceed a certain prescribed limit.

A general computer program 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 initialization 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

$$h_x = (RQ^n)_x = (RQ^{n-1})_P Q_x = (R'Q)_x \qquad (12.9)$$

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 nonlinear 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 and Gupta 2006). Thus, for the i^{th} iteration,

$$_{i+1}Q_{xa} = \frac{_{i}Q_{xa} + {}_{i}Q_{xo}}{2} \qquad (12.10)$$

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 initialization 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 $_t\Delta H_i$, $_t\Delta H_j$, and $_t\Delta Q_x$ 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

$$\left({}_tH_{oi} + {}_t\Delta H_i\right) - \left({}_tH_{oj} + {}_t\Delta H_j\right) = R_{ox}\ {}_tQ_{ox}^{n} + nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}{}_t\Delta Q_x, x = 1,\ldots,X \qquad (12.11)$$

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

$$_{t+1}H_i - {}_{t+1}H_j - nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}{}_t\Delta Q_x = R_{ox}\ {}_tQ_{ox}^{n}, x = 1,\ldots,X \qquad (12.12)$$

Subtracting $nR_{ox}\ {}_tQ_{ox}^{n}$ from either side

$$_{t+1}H_i - {}_{t+1}H_j - nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}\left({}_tQ_{ox} + {}_t\Delta Q_x\right) = (1-n)R_{ox}\ {}_tQ_{ox}^{n}, x = 1,\ldots,X \qquad (12.13)$$

Replacing, $_tQ_{ox}$ + $_t\Delta Q_x$ by $_{t+1}Q_x$

$$_{t+1}H_i - {}_{t+1}H_j - \left(nR_{ox}\left|{}_tQ_{ox}\right|^{n-1}\right){}_{t+1}Q_x = (1-n)R_{ox}\ {}_tQ_{ox}^{n}, x = 1,\ldots,X \qquad (12.14)$$

Equation (12.14) 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

$$\sum_{x \text{ connected to } j} {}_{t+1}Q_x + q_{oj} = 0, \; j = M+1,\ldots,M+N \qquad (12.15)$$

which are *N* linear equations. Solution of set of Equations. (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 is given by

$$\mathbf{{}_{t+1}H = -[A_{21}(NA_{11})^{-1} A_{12}]^{-1}[A_{21}(NA_{11})^{-1}(A_{11}\, {}_tQ + A_{10}\, H_0)-(A_{21}\, {}_tQ - q_0)]} \qquad (12.16)$$

$$\mathbf{{}_{t+1}Q = (I - N^{-1})\, {}_tQ - [\, N^{-1}\, A_{11}^{-1}\, (A_{12}\, {}_{t+1}H + A_{10}\, H_0)]} \qquad (12.17)$$

In which **N** and $\mathbf{A_{11}}$ are diagonal matrix of size (X, X); $\mathbf{A_{12}} = \mathbf{A_{21}^T}$ 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 initialization. The hydraulic solver EPANET is based on Gradient Method (Rossman 2001). An illustrative example is provided in Annexure 12.5. to show the step-wise calculation of nodal flows and heads in a simple looped network using the gradient method.

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 as 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 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 as a *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; and at available pressure just equal to the minimum head, outflows are considered between 0 and required demand, and obtained using optimization. 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 having different pressure requirements due to their locations, NHFR of Wagner et al (1988) is more appropriate than of Bhave (1981).

As suggested by Bhave (1981), the available flow at demand node *j* may be characterized as follows:

$$q_j^{\text{avl}} = q_j^{\text{req}} \text{ (adequate flow), if } H_j^{\text{avl}} > H_j^{\min} \tag{12.18}$$

$$0 \le q_j^{\text{avl}} \le q_j^{\text{req}} \text{ (no flow, partial flow or adequate flow), if } H_j^{\text{avl}} = H_j^{\min} \tag{12.19}$$

$$q_j^{\text{avl}} = 0 \text{ (no flow), if } H_j^{\text{avl}} < H_j^{\min} \tag{12.20}$$

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:

$$q_j^{avl} = q_j^{req}\left(\frac{H_j^{avl} - H_j^{min}}{H_j^{des} - H_j^{min}}\right)^{\frac{1}{n_j}}, \text{ if } H_j^{min} < H_j^{avl} < H_j^{des} \tag{12.21}$$

where n_j is a coefficient. Different values between 1 and 2 have been suggested in the literature (Bhave and Gupta 2006).

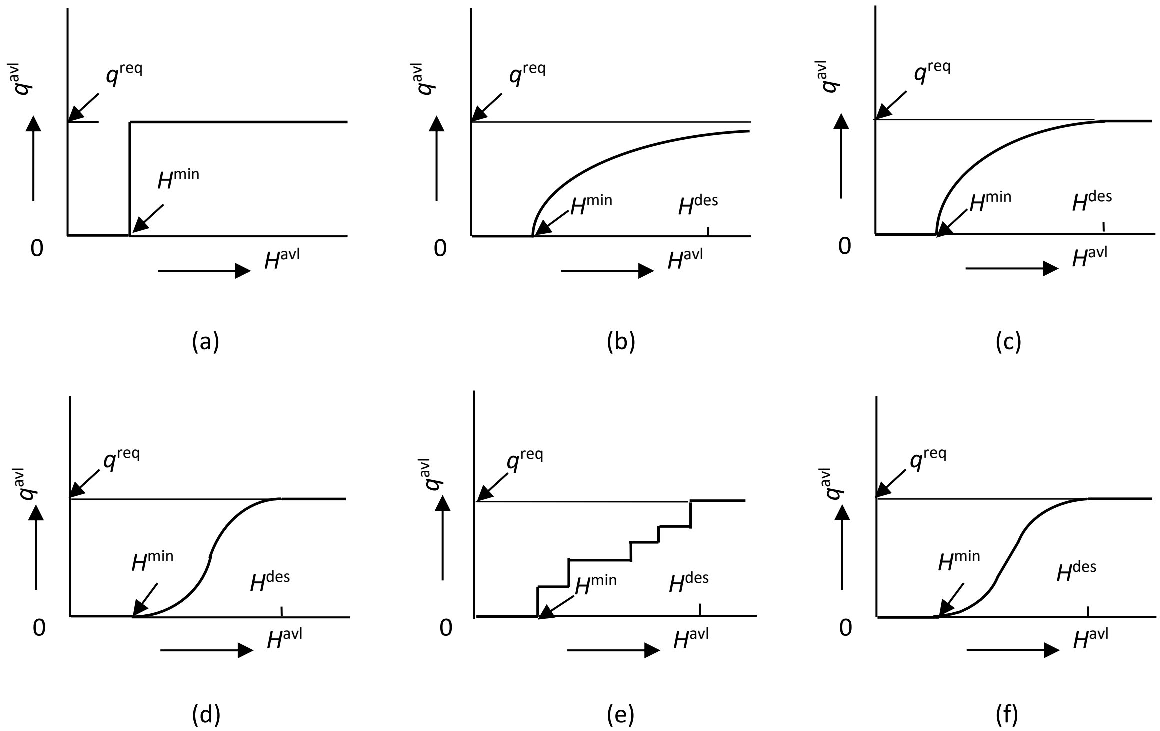

Figure 12.3: Nodal head-flow Relationships: (a) Bhave (1981) (b) Germanopoulos (1985) (c) Wagner et al. (1988) (d) Fujiwara & Ganesharajah (1993) (e) Kalungi and Tanyimboh (2003) (f) Tanyimboh and Templeman (2010)

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 in 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.

B. Dynamic Analysis or Extended Period Simulation

The NHA (or DDA) and NFA (or PDA) gives instantaneous picture (snap shot) 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 as 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 nonlinear 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. For 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 design of pipe WDS essentially involves determination of location and sizes of different components which will meet the physical and operation 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. As the pipes have life 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 design of a WDS is to minimize the cost and 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 water distribution system. 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 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 judgement 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 optimization 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 that they assume pipe diameter as continuous variable and at the end of optimization 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 for large practical size networks. Even though several algorithms were developed and tested for their efficacy on small networks, none of them were observed to handle large practical size networks with all complexities. The optimization 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 Ver. 4 developed by UNICEF is freely available in public domain that 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 includes Genetic Algorithm, Simulated Annealing, Particle swarm optimization, 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 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 optimization 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 program 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 capitalized 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 these types of measures reduce the number of evolutions, the application of methodologies to large practical size 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 equalization, and leakage reduction in a framework of multi-objective design problems.

12.7.2 Optimization of Pipes in Operational Zones

In most of the cases, some pipes in distribution system are existing and all operational zones are to be reengineered with increased nodal demands, peaking factor, and revised norm of residual nodal head.

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. So 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 Levels (LSL)s of ESRs are assigned. The required diameters are assigned to each pipe, as per the experience and judgement 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) It is to be ensured that the boundaries of all operational zone are designed optimally, i.e., ESRs/GSRs do not get empty or get overflowing.

(d) 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.

(e) 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 operational zone. The logic is shown in Figure 12.4.

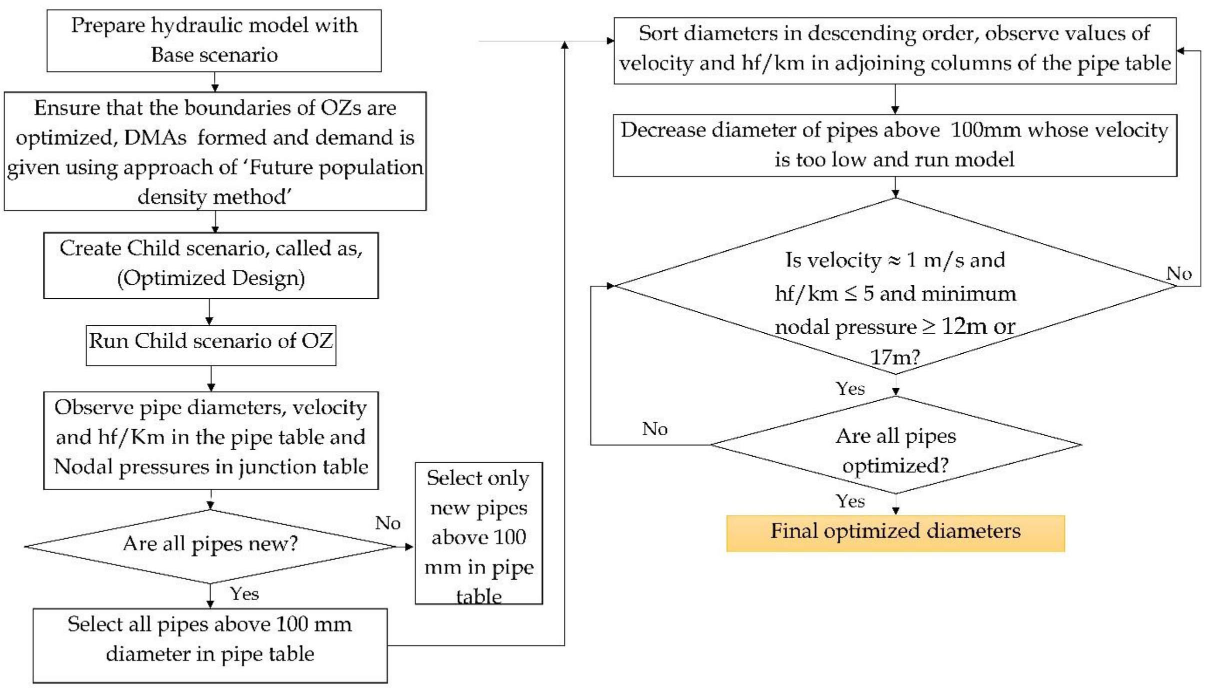

Figure 12.4: Iterative process of optimizing diameters.
(This iterative process is not performed by network software and is to be done by the designer)

(f) Run the model. In each iteration, observe pipe diameters, velocity and hf (m/km) in the pipe table and nodal pressures in junction table.

(g) Select only new pipes in pipe table. Following steps are taken for new pipes keeping existing pipes undisturbed.
(h) Sort diameters in descending order, observe values of velocity and hf (m/km) in adjoining columns of the pipe table.
(i) Decrease diameters of the pipes in which velocities are too low and whose diameter > 100mm and again run model.
(j) 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 5 m/Km, and minimum nodal pressure is also more than or equal to residual nodal head as per norm, the steps are repeated. (Sometimes diameter needs to be increased).
(k) The process is repeated for all the new pipes whose diameters are more than 100mm, till we get all optimized diameters.
(l) It can be easily seen that the diameters of existing pipes are retained and only the new pipes are optimized.
(m) It is experienced that one operational zone can be optimized within half hour using this technique.

12.7.3 Rehabilitation of Water Distribution Systems

The efficient and effective management of existing water distribution system (WDS) face challenges due to non-mapping of existing infrastructure, aging 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 & variations in mid & long-term perceptions.

Modelling solutions therefore should be used to assess the rehabilitation of older water distribution system. 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, to replace 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 are incorporated in Annexure 12.2.

If the existing intermittent water supply system with existing ESRs of a project area is to be converted into 24x7 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 Central Building Research Institute (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) Down-take supply system with or without sump and pump.

If the pressures near the premises are adequate to supply water for 24 (twenty four) 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 down-take 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 byelaws.

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) on the main, which is throttled sufficiently to deliver the required supply at the minimum residual pressure of 21m or 15m as the case may be. The supply is also controlled by a stop cock at the beginning of the service pipe - A meter is to be installed beyond the stop cock 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 stop cock 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, 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 theadvantage of low cost and high strength. They suffer from the disadvantage of short life 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^{o}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 stop cock 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 & 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 3 meters of horizontal separation from an existing or prospective drain or sewage line. If the 3 meters 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 back filling.

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 (24x7) 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 *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 softwares 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 extended period simulation 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 Method, Linear Theory Method, Newton- Raphson Method and Global Gradient method are evolved. Amongst them, Global Gradient method is widely used in the computation engine of all the software.

12.10.1 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. Some software products contain scenario manager (analysis “what if” conditions) that helps the user to define and manage of specific model runs. Additional scenarios can be developed to test the sensitivity of the system to variation in the model parameters that are known with certainty.

12.10.2 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

12.10.3 EPANET Freeware Software

EPANET 2.0 is software application used round the world to model water distribution system. 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, EPANET2.2, has capability 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.

12.10.4 Developing a Basic Network Model

The basic network model should first be characterized. 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.

12.10.5 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.

12.10.6 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 data base 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 synchronize 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 digitizing 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 12.5. Resulting hydraulic model uses combined capabilities of both the softwares.

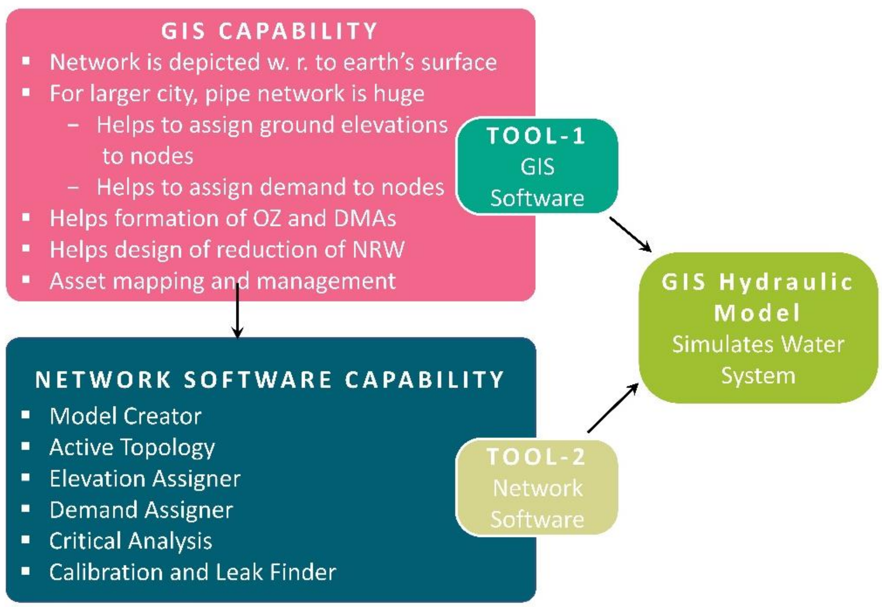

Figure 12.5: Integration with GIS

12.10.7 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) Digitization of water supply components (iii) Landmarks (iv) Existing water infrastructure and (v) Contours

i. **A Satellite Image:** of 0.5 m resolution satellite image is obtained from the National Remote Sensing Authority (NRSA) Hyderabad. Alternatively, most of the GIS softwares provides online satellite image which can be used. The features such as roads, building footprints, water bodies should be digitized.
ii. **Digitization:** It 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 digitization, information is obtained which makes it easier to preserve, access, and

share. There are number of ways of making digitization which are: (a) From hardcopy of Existing Map (b) From Google Earth (c) From Online Service Satellite Image and (d) From Open Street Map (Freeware).

iii. **Landmarks:** Open the GIS software and add imagery to it. Landmarks can be created by selecting– Boundaries and Places, Transportation besides that of Imagery. Shape file of landmarks then can be created by digitizing the landmarks say, temple, road name etc and then exporting to shapefile. Alternatively, landmarks can be created directly from the Google Earth.

iv. **Existing Pipes:** For creation of hydraulic model, existing pipelines need to be identified by making *condition assessment* exercise of the present infrastructure. Emphasis should be given to use existing pipe network.

v. **Contours:** Hydraulic models use elevation data to convert heads to pressure. Actual pipe elevations should be used to establish the correct hydraulic grade line (HGL). Elevations are assigned to each node in a network where pressure information is required. Various techniques are employed to determine elevation information like GIS based contours include the following:

(a) Topographical maps
(b) Total Station Survey with built-in facility of recording Latitude and Longitude of the point under consideration
(c) 3D stereo paired satellite image used in conjunction with photogrammetric softwares
(d) Flying Drones
(e) LiDAR (Light Detection and Ranging)
(f) Digital elevation models (DEM) with minimum interval of contour
(g) Global positioning system (GPS)

GIS base 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 shapefiles.

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 two (Planning) which can be used.

c) **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.

d) **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 operational zone. Hence demand of operational zone 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%.

e) **Creation of Active Topology:** Using shapefile 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.

f) **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 12.5). Using GIS, values of ground elevations and demand of water are given to each node of distribution system.

g) **Assigning Ground Elevations to Nodes:** Assigning ground elevations to the nodes is described in Figure 12.6. 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 coordinate system for both the layers of contours and pipe nodes.

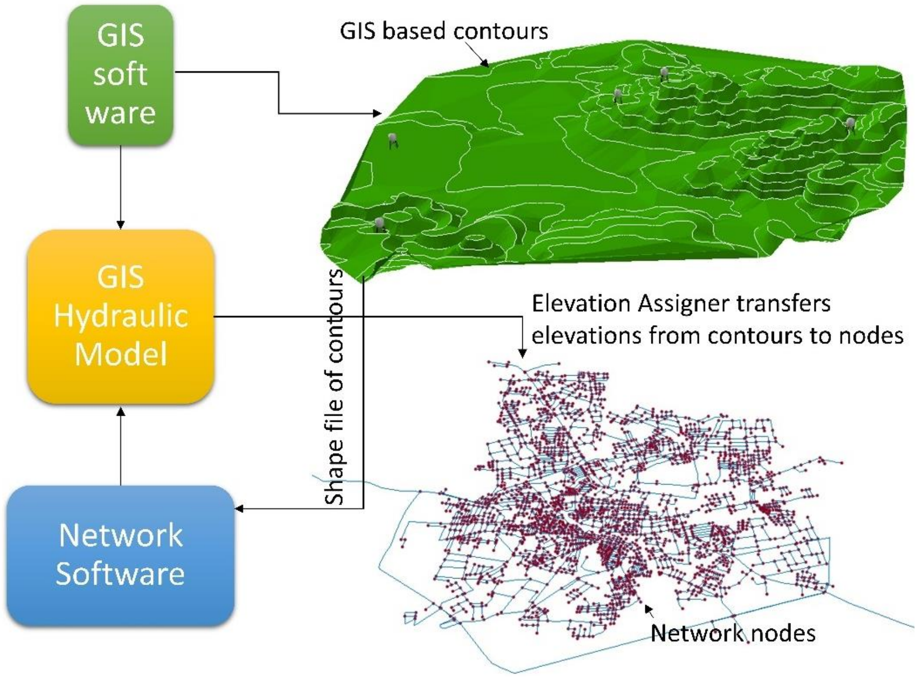

Figure 12.6: Assigning elevations to nodes using network software

12.10.8 Water Demand Inputs

Water demand is the driving force behind the operation of a water distribution system. 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 2.3 in Chapter 2). 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 12.7), and then forming polygons around each node (Figure 12.8).

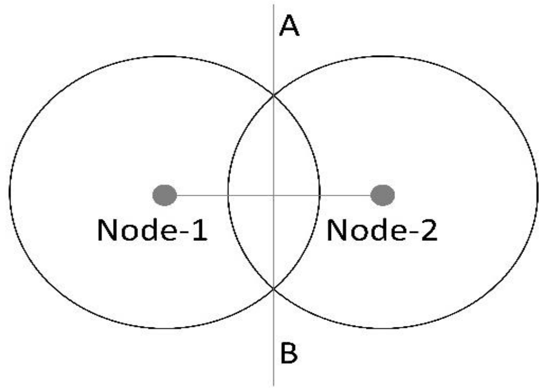

Figure 12.7: Perpendicular bisector of a line joining the two adjacent nodes

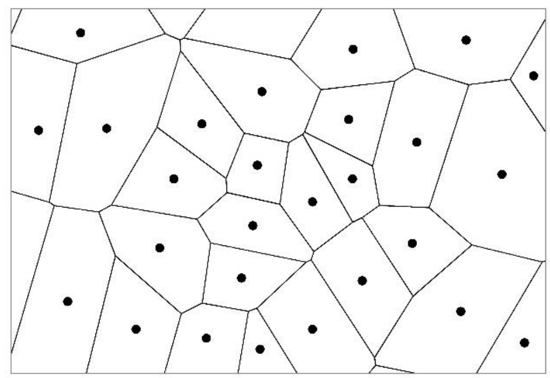

Figure 12.8: Polygons formed around each node

Second important task is to assign the demands to each node. It is carried out by the tool called as, *Demand Assigner* (Figure 12.9). 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.

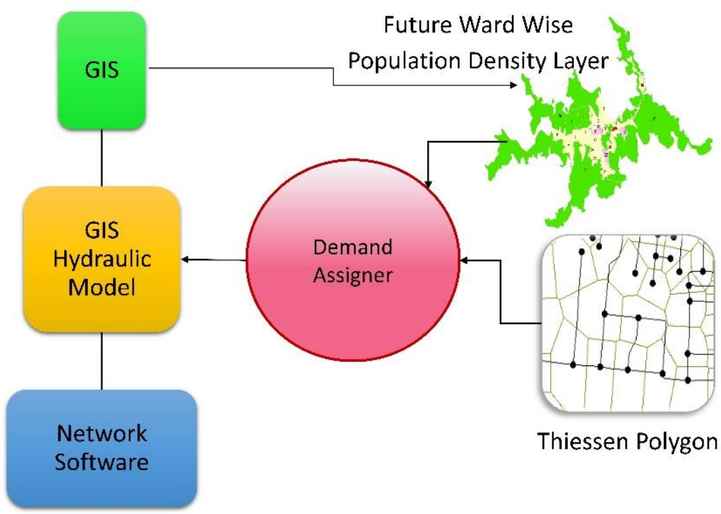

Figure 12.9: Assigning demand to nodes

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 operational zones of the existing service reservoirs should be decided utilizing the logic as discussed in the subsequent paras. However, after marking the optimized 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. Operational zones (OZ) can be formed for each individual reservoir depending upon the situations of reservoirs. Operational zones 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.

Operational zone (OZ) is the jurisdiction of each tank to serve water supply. Performance of distribution of water depends on size of operational zone of tank. A schematic of the operation zone with DMAs is shown in Figure 12.10.

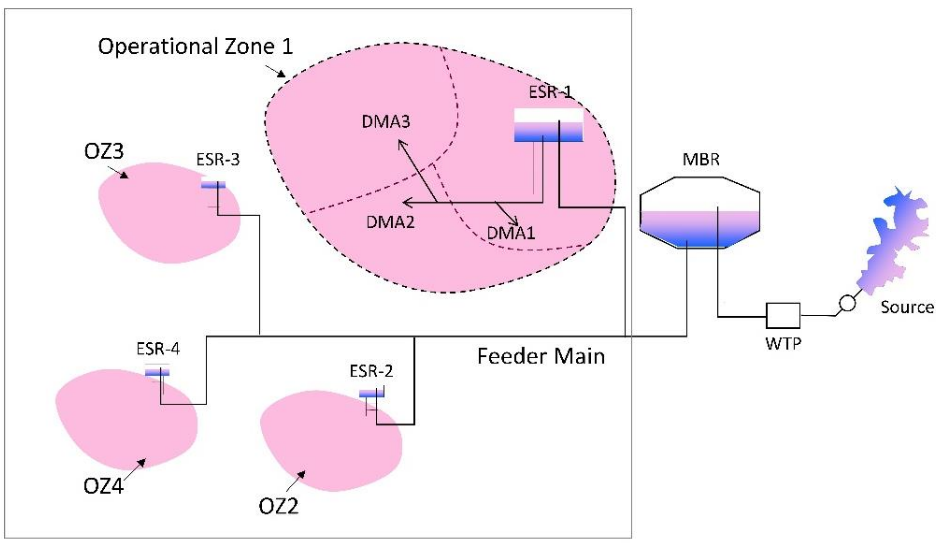

Figure 12.10: Operational zone with DMAs

12.11.1 Design Criteria for Operational Zones

For the approach of 24x7 system, following are design criterions for operational zones 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 operational zone 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.

12.11.2 Developing Operational Zone 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 Operational Zone

To begin with, it has to be borne in mind that if excessive capacity of existing ESR remains unutilized, 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 = Lowest Supply Level (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 figure of 5m 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 12.11.

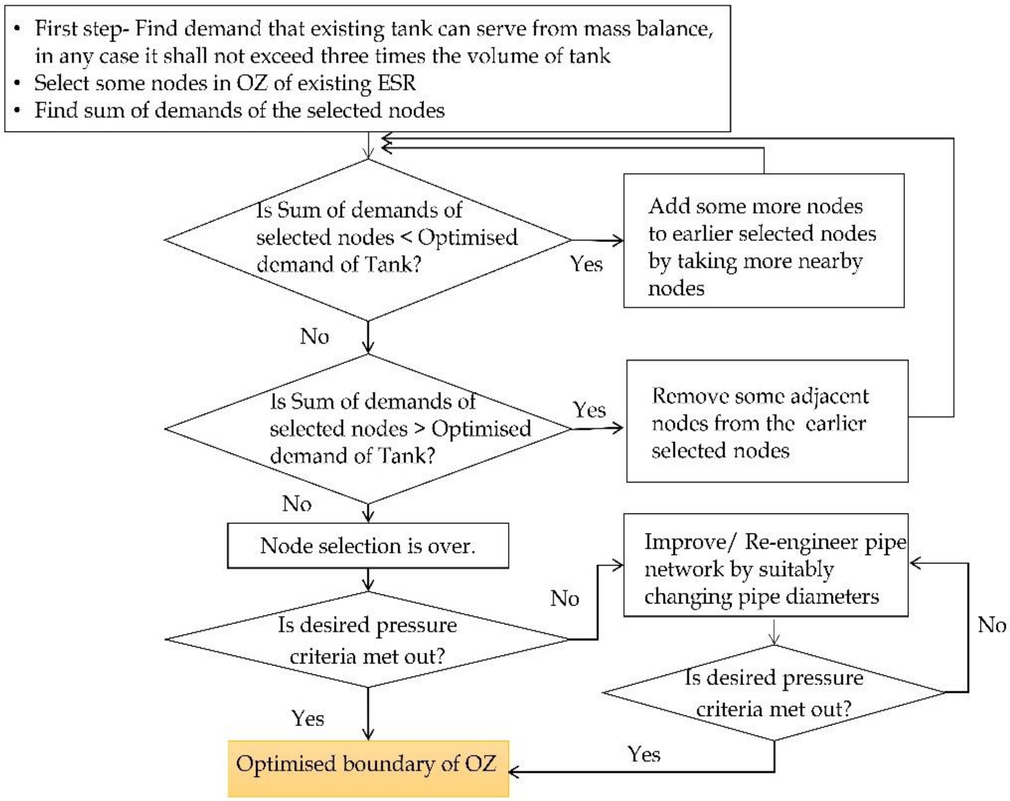

Figure 12.11: Logic for fixing boundary of operational zone of existing tank (K.S. Bhole, 2015)

A hydraulic model should be created using the existing pipe as well as the new pipes. Suppose the network is as shown in Figure 12.12(a). Objective is to decide and fix the extent of the operational zone of this existing tank. Process of selection of nodes to fix boundary of operational zone of existing tank is shown in Figure 12.12.

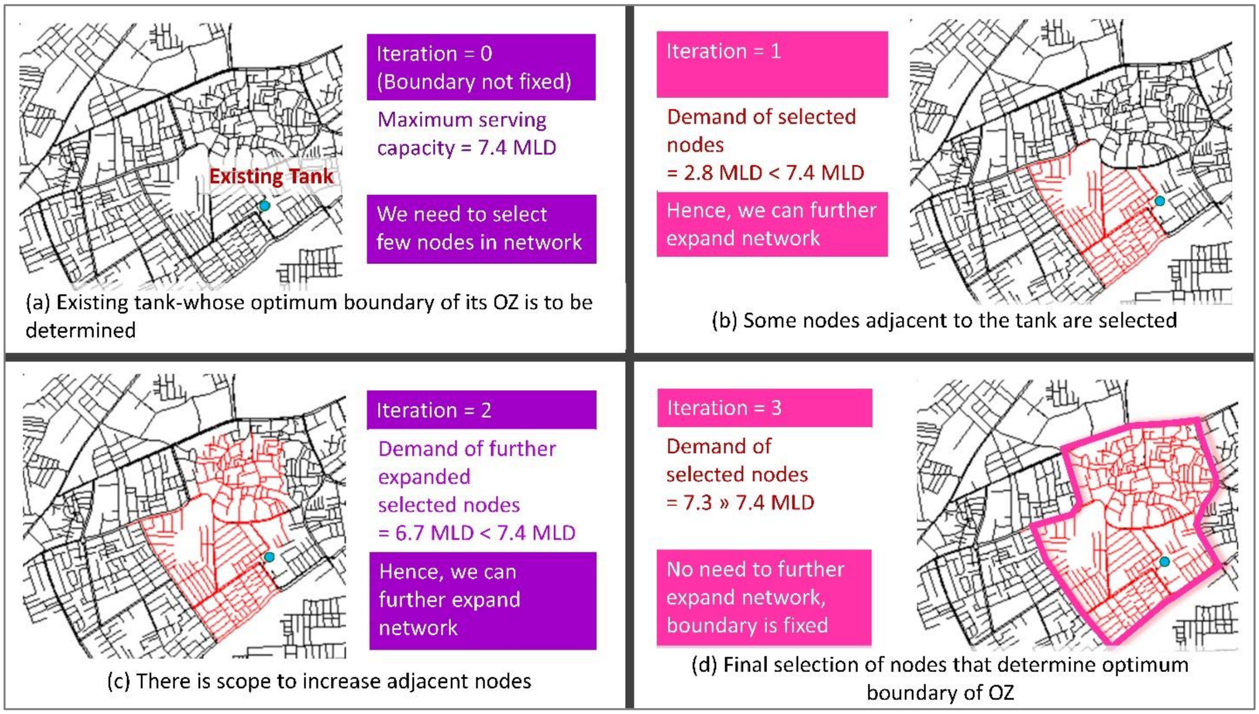

Figure 12.12: Process of selection of nodes to fix boundary of operational zone of existing tank

The process starts with iteration 0. Let the optimum serving demand of the existing tank is 7.4 MLD. (Figure 12.12 (a). In iteration 1, some nodes in the vicinity of the existing tank are selected as shown in red colour in Figure 12.12 (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 12.11. 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 operational zones are not hydraulically discrete, any connecting pipe between the two operational zones 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 operational zone to other.

12.11.4 Optimization 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 optimization techniques to optimize 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 program or on the top of hydraulic models. Optimization techniques that have been studied include methods such Genetic Algorithms, Linear Programming, Non-Linear Programming etc.

Some of the network softwares have a built-in facility of optimizer tools. But those softwares are costlier. Many designers face difficulty in understanding and using this tool while designing their projects for pipeline optimization. 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 softwares 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 softwares depict 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.

Objective is, therefore, to discuss a novel methodology of designing pipes by using only the hydraulic model without any optimizer software. This methodology has been discussed at length in the paper (IWWA, 2020). The method requires any software by which we can prepare hydraulic model. For example, EPANET software which can be downloaded without any cost from internet, can be referred as “network software.”

Design Principles for Optimizing Diameters

While designing network for operational zone/ DMA, designer assigns the required data of lowest supply level (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 Lowest Supply Level 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 operational zone 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 center of operational zone fulfilling both conditions, this type of arrangement is seldom possible. Try to fulfil them to the extent possible.
6) Lowest Supply Level of water of ESR should be equal to highest ground level in operational zone + minimum residual head + head loss for getting desired velocity.

7) Velocity should not be less than 0.3 m/s in all diameters above 80/100mm.
8) The minimum diameter in the distribution system of city/ town should be 100 mm for class I cities and other 80mm.

12.12 District Metered Area (DMA)

UK Water Authorities Association in 1980 introduced the concept of District Metered Area (DMA). They used DMA management technology for monitoring leakages in water distribution networks. The water distribution system before and after advent of DMA is shown in Table 12.1.

Table 12.1: Water distribution system before and after advent of DMAs

S. No.	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 as *active leakage* control. DMA is therefore the building block of 24x7 continuous water supply.

A District Metered Area (DMA) is a sub zone within operational zone of a water distribution network that can be hydraulically isolated and for which water consumption is measured using water meters. Bulk flow metres 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 prioritize leak identification and repair program 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 12.13 and a single typical hydraulically discrete DMA is shown in Figure 12.14.

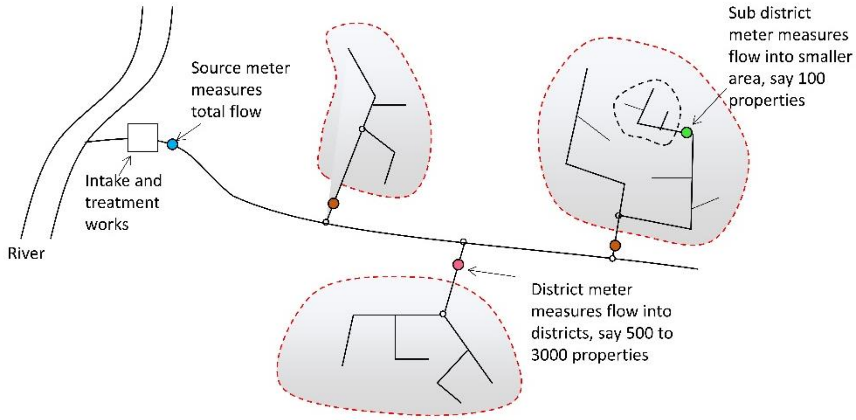

Figure 12.13: Typical DMA Structure

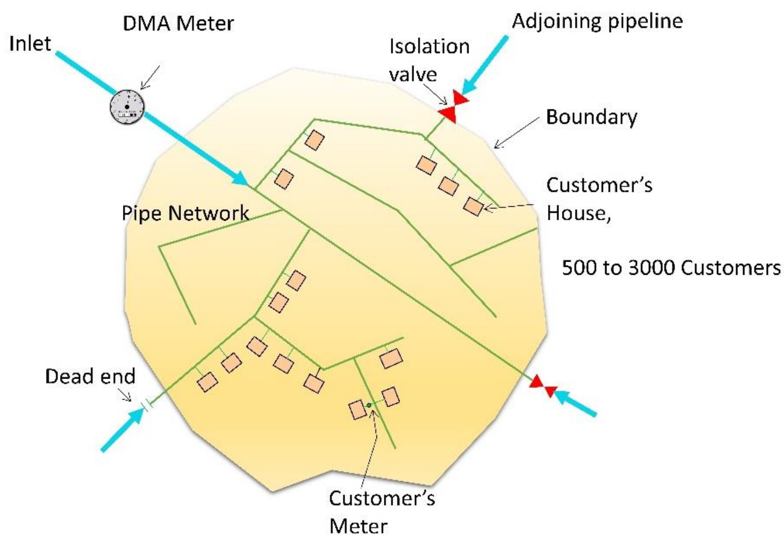

Figure 12.14: Typical hydraulically discrete single DMA

12.12.1 Types of DMAs

DMAs are categorized 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 12.15.

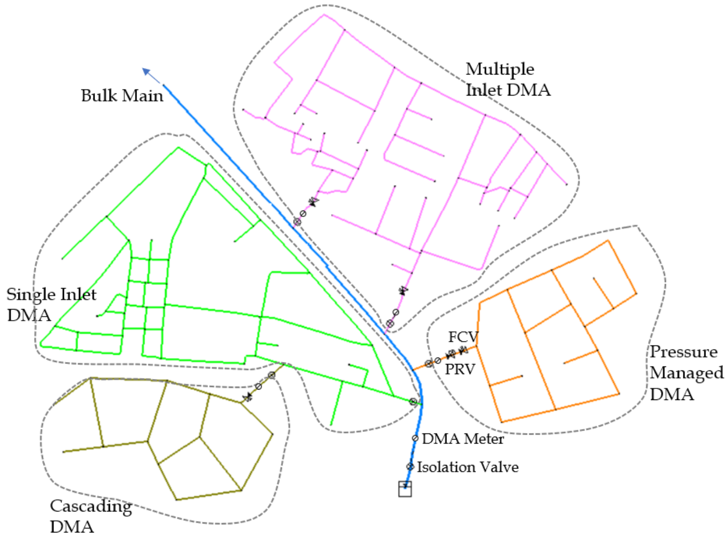

Figure 12.15: Types of DMA

(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.

12.12.2 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 District Metered Area (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 operational zone, 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 operational zone. This is also necessary due to the scarcity of suitable sites for additional service reservoirs. DMAs are also designed within the operational zone 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 pressure reducing valve (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 as *clustering*; (b) identification of the pipes where isolation valves and flow meters needs to be installed, which is called as *sectorization*; 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 Gephi (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, WTP's, 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. Sectorization Process

The second step of DMA design is to identify the open/closed status of boundary pipes. It is also called sectorization of network. The basic idea of sectorization 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 flow meter can be added only at the start of cascading DMA. Conventionally, sectorization is achieved by minimizing the cost of flow meters 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 optimization 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 optimization 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.

12.12.3 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
d) previous leakage control technique (like, ex-waste meter districts)
e) individual water Agency/ Board preference (like, discrimination of service pipe bursts, ease of location survey)
f) 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 12.16) should be carried out to validate hydraulically discreteness of the DMA.

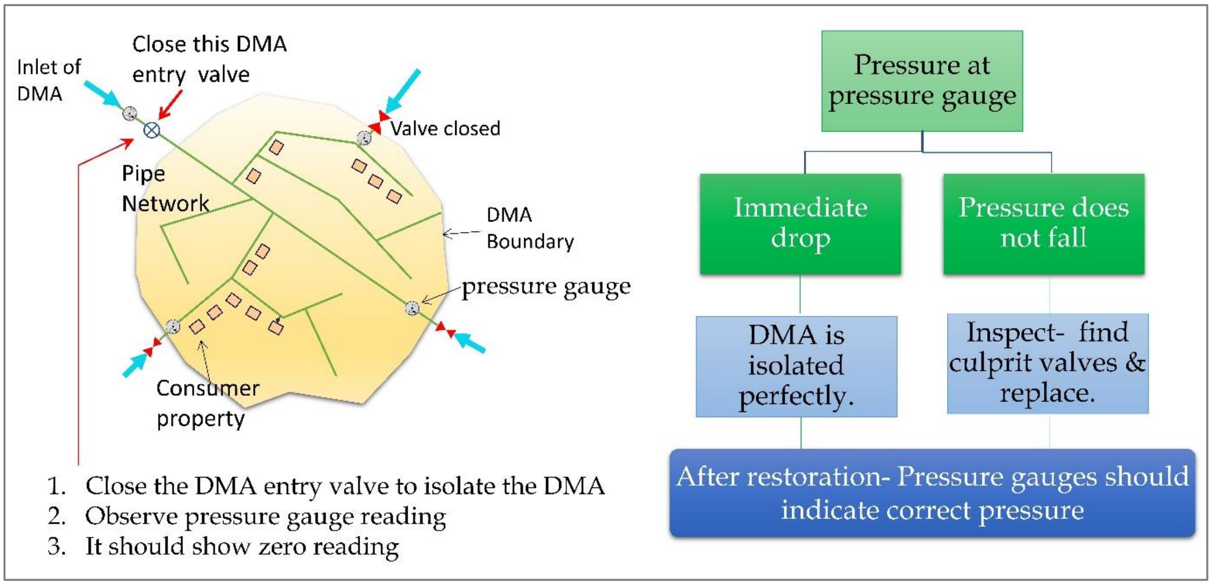

Figure 12.16: Zero pressure test

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 operational zones and DMAs. To start with the topology of the corresponding operational zone is activated as shown in Figure 12.17. De-activated nodes and pipes are shown in red color.

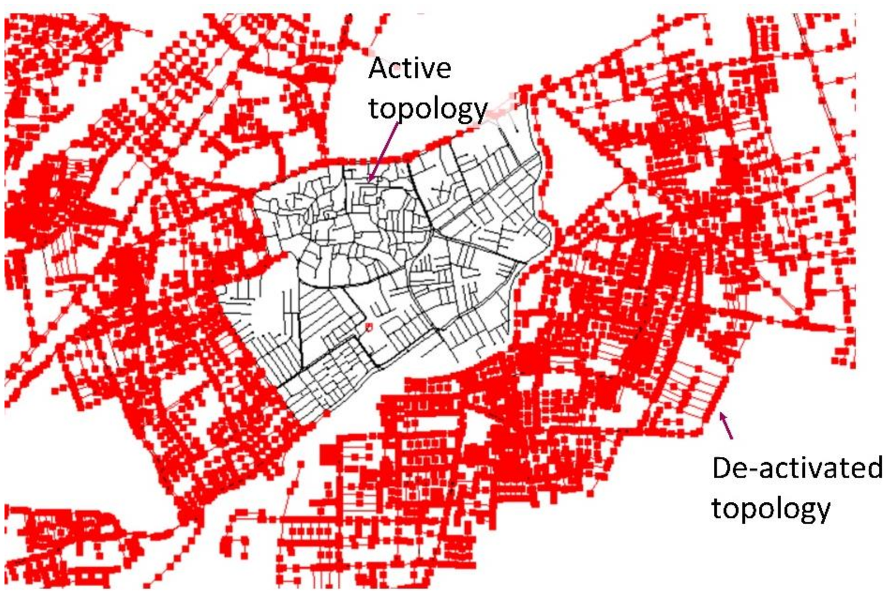

Figure 12.17: Activated network of operational zone

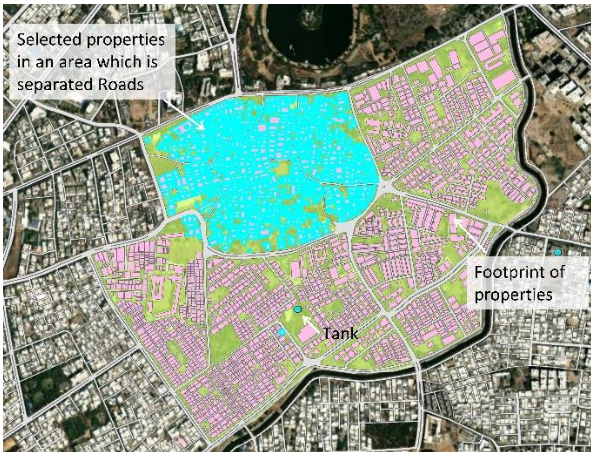

Figure 12.18: Selected nodes in one area bounded by the roads

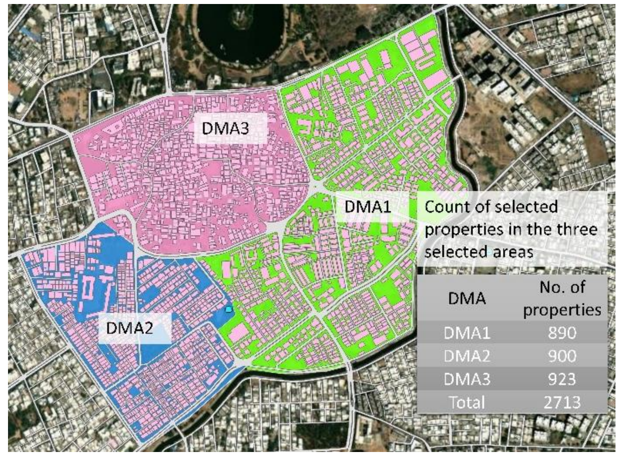

Figure 12.19: Nodes in all the three areas

The extent of operational zone (Figure 12.17) seems to be large and hence about three DMAs are expected. Depending upon the size of the operational zone, the number of DMAs to be created shall be anticipated. To begin with, three DMA areas in the activated operational zone are selected by colour coding. It is required to compute the number of properties in these three selected areas. Shapefile of the three areas is created. Using GIS tool, the number of properties in one of the areas (Figure 12.18) are selected and counted. In this way, the number of properties in all the three areas (Figure 12.19) are counted.

Alternatively, one field is added in the *attribute table* of the shapefile of properties in all the three areas. The shapefile of the properties is then *spatially* joined with the shapefile 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 labeled. By summarizing 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 12.19.

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 and if VFD pump is proposed on the outlet of the service tank, then all DMAs should be fed by branch pipelines starting from outlet of ESR in operational zone (Figure 12.20).

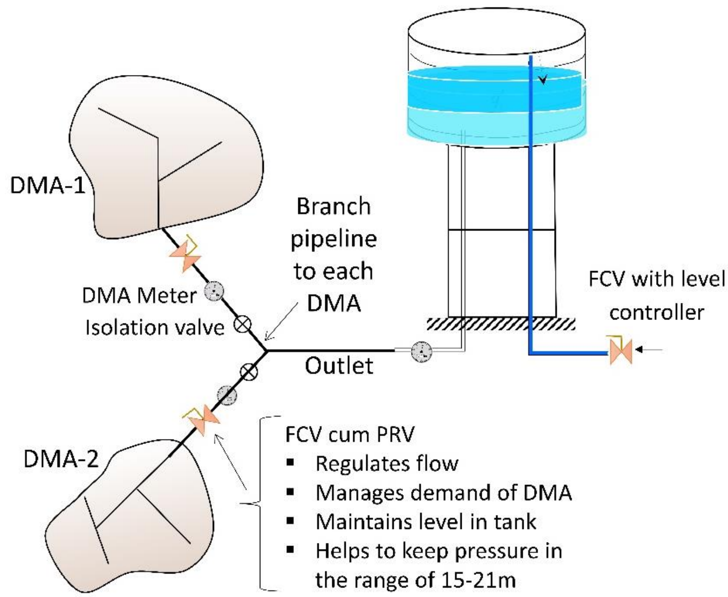

Figure 12.20: Separate inlets of each DMA

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.

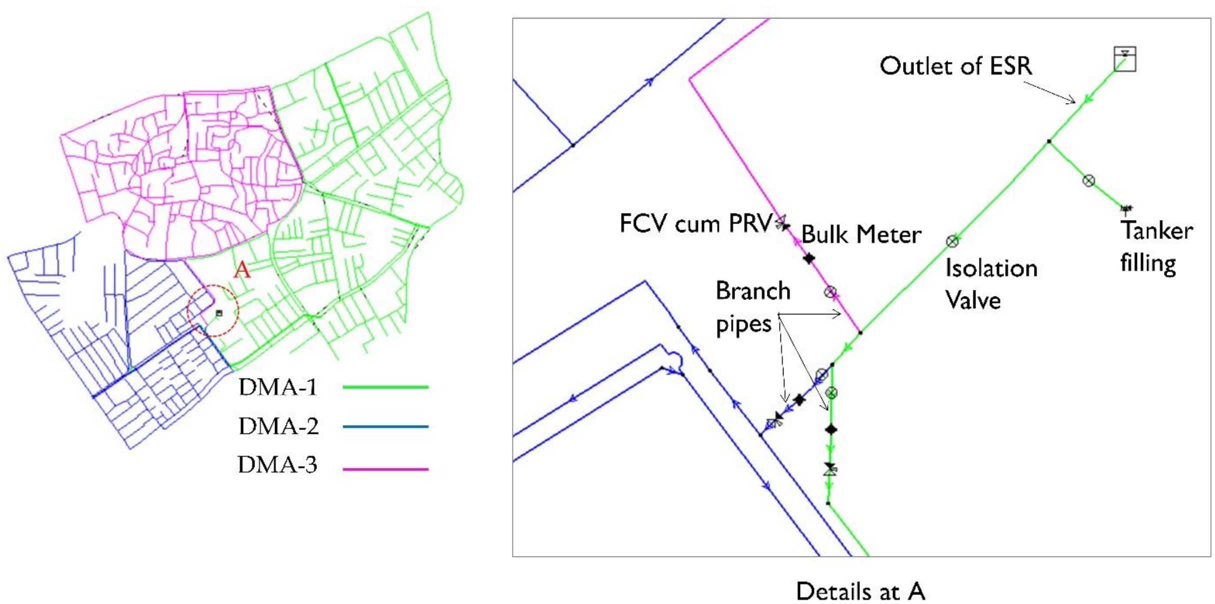

Figure 12.21: Separate branch pipes for entry of each DMA

The arrangement of separate branch pipes for entry to each DMA is shown in Figure 12.21.

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 operational zones 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

Operational zones 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 operational zone 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 operational zone 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 24x7 System. It is achieved by Whole-to-Part approach, in which two stages are involved- (i) from MBR to ESRs and (ii) from ESR to DMAs and (iii) Equitable Pressures within DMA.

(i) **From MBR to ESRs:** In this stage, Master Balancing Reservoir (MBR) supplies water to different ESRs as shown in Figure 12.22.

   Equalization 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.

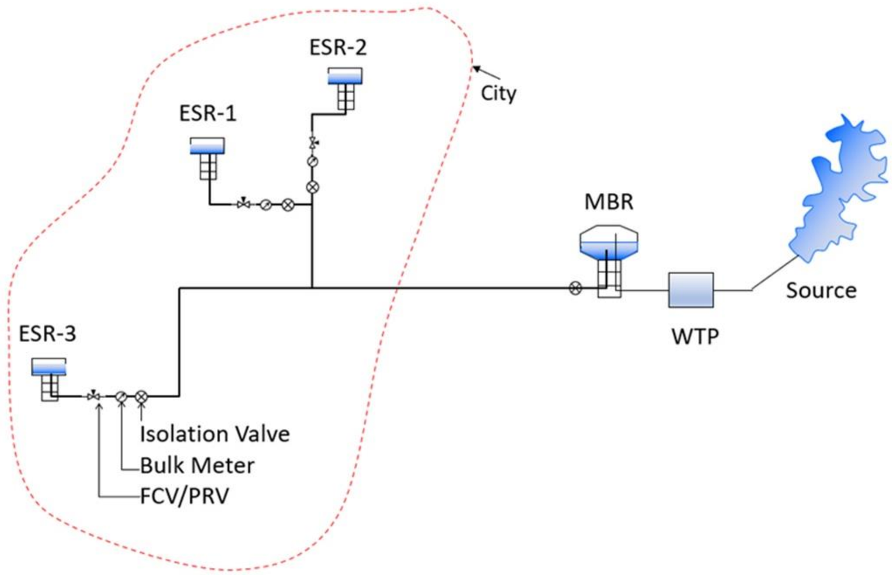

Figure 12.22: Separate branch pipes for entry of each DMA

In addition to equalization of heads at ESRs, the inlet of each tank should be provided with isolation valve, bulk meter and then Flow Control Valve (FCV)/ Pressure Reducing Valve (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 12.22). 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 operational zone 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 shall be normally 4 to 5m.

12.14.2 Improving nodal pressure to 21m

Hydraulic Model of 24x7 Water Supply Using VFD Pump

The residual nodal pressures are recommended to be in the range of 15-21 m at critical node of the distribution network. However, in India, many service reservoirs have staging height less than 12m and may not be able to meet the pressure requirement by any change in pipe sizes. Hence, for 24x7 water supply instead of constant speed pumps, direct feeding of networks is recommended using Variable Frequency Drive (VFD) operated pumps adopting smart control philosophy. A new network can be easily designed with VFD pump installed at clear water sump of Water Treatment Plant (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 15-21 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 15m to 21m. 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, water distribution networks (WDNs) are usually designed to achieve a residual head of 7 to 12 m for supply to single and double storied 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 15 to 21 m at all nodes throughout the day. A combined pumping and gravity network are preferrable 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. Variable frequency drive (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 elevated service reservoir (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 District Meter

S. No.	Gravity Feed Network	Direct Feed Network
		Areas (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.
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 24x7 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 Variable Frequency Drive (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, higher the frequency, 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 1 second, then the frequency of the given wave is 50 Hz. In India, the value of frequency is 50Hz. This value is standardized 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,

$$N = RPM = \frac{120f}{P} \qquad (12.22)$$

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 Figure 12.23. Starting point of the pump curve is at the point of zero flow, at which the pressure head generated by the pump is called as *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 m3/hr) at minimum rated speed Nmin (in RPM), and Qmax = maximum flow (LPS or LPM or m3/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:

$$\frac{Q_2}{Q_1} = \frac{N_2}{N_1} \tag{12.23}$$

$$\frac{H_2}{H_1} = \left(\frac{N_2}{N_1}\right)^2 \tag{12.24}$$

$$\frac{P_{i2}}{P_{i1}} = \left(\frac{N_2}{N_1}\right)^3 \tag{12.25}$$

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.

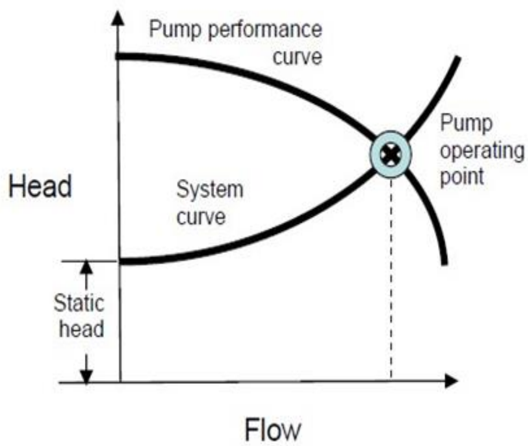

Figure 12.23: Operating point of pump

System Head for a simple pumping main can be defined in mathematical form as

$H_s + (R * Q^n)$ (12.26)

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 12.23) which is called as 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 12.23, 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 as 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.

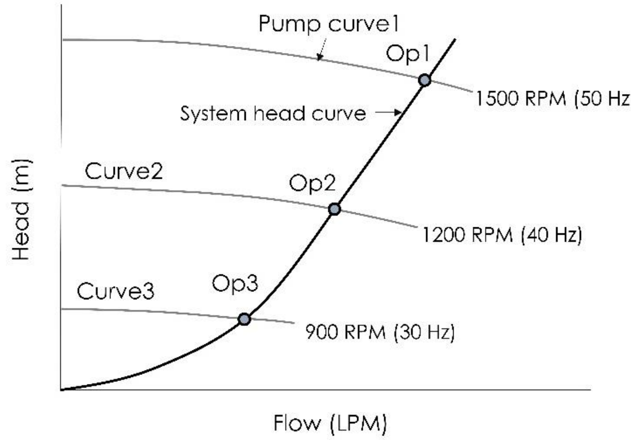

Figure 12.24: Attainting operating points on system curve

Figure 12.24 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 12.24, 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 mid night 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 operational zone of the Ayodhya city using VFD pumps is discussed as shown in Annexure 2.4.

Excessive Nodal Pressures

The design of operational zone is carried out for a nodal pressure of 21m. 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 12.25, due to low elevations in the terrain, some nodes in DMA 2 have nodal pressures even more than 40m. Authority encounter the 'tail end problem' which causes a major obstacle in ensuring equal water supply at every tap.

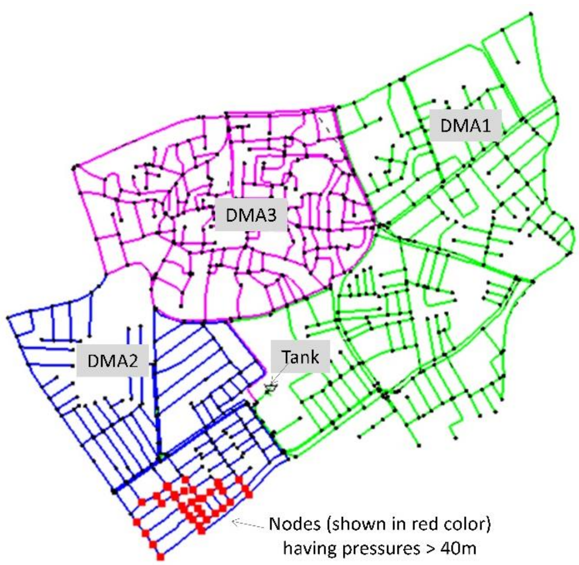

Figure 12.25: Higher nodal pressures

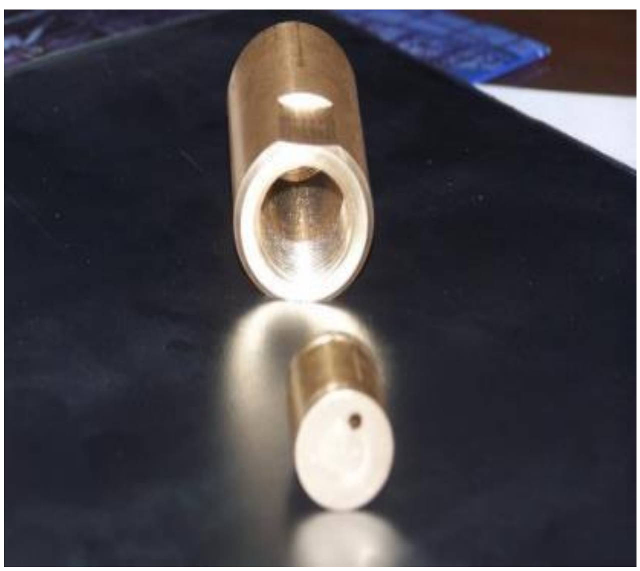

Figure 12.26: Pressure and Flow Rate Reducing Valve [PFRV]

To resolve this problem, it is proposed to install (Figure 12.26) Pressure and Flow Rate Reducing Valve [PFRV] in the water distribution system. 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 reduce the water pressure (dynamic + static) and flow rate; of a flowing water, simultaneously, in almost equal proportions; and helps in resolving the 'tail end problem', saving consumers from water splashes and in reducing the 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 02 - 03 kg/cm^2 to receive a water flow rate of about 05-08 litre/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 is considered to have a larger life expectancy than a PRV (Pressure Reducing Valve). 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 water distribution networks, 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 21m 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 pressure reducing valve (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, a large number of 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 12.27 and with PRV in Figure 12.28.

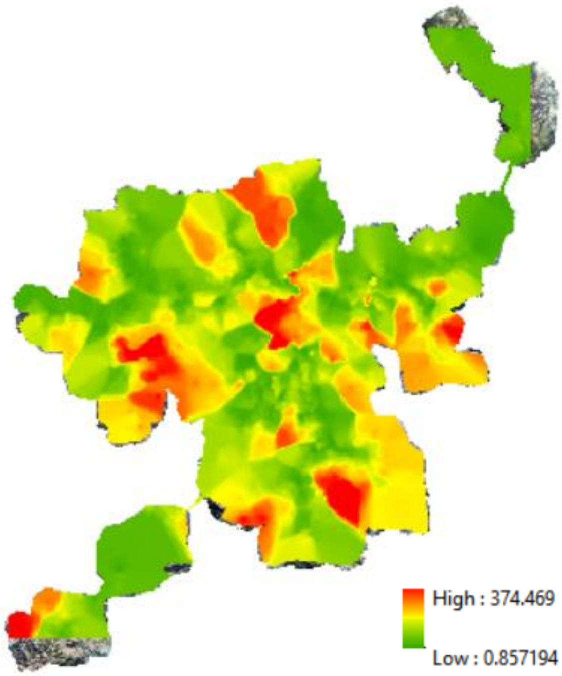

Figure 12.27: Without PRV: Pressures in hilly area

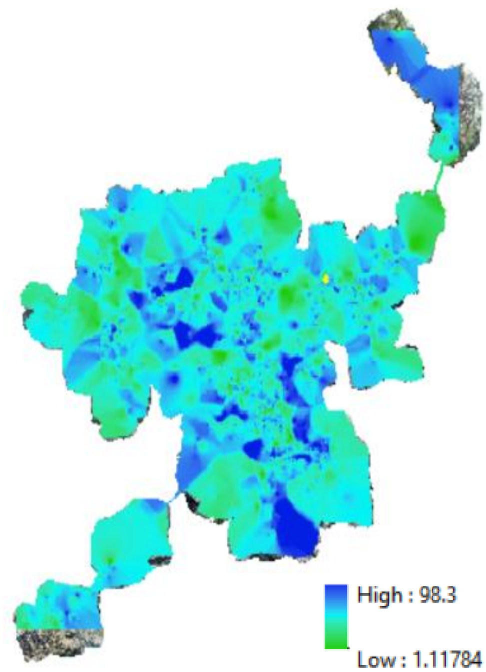

Figure 12.28: With PRV: Pressures in hilly area

From Figure 12.27 and Figure 12.28, 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 maintained in the range of 20 to 98 m. Thus, PRVs play a very important role in pressure management of the hilly areas. However, it is required to further reduce the pressures up to 21m.

Direct acting PRVs: Direct acting PRVs are used to reduce pressures to 21m. 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 Non-revenue Water (NRW) which is defined as,

$$NRW = \frac{\text{Water put into system - Total water billed}}{\text{Water put into system}} \text{ x } 100 \qquad (12.27)$$

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 program 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 programs are often promoted to meet the ever-increasing demand. In short, the utility enters a vicious cycle (Figure 12.29) that does not address the core problem.

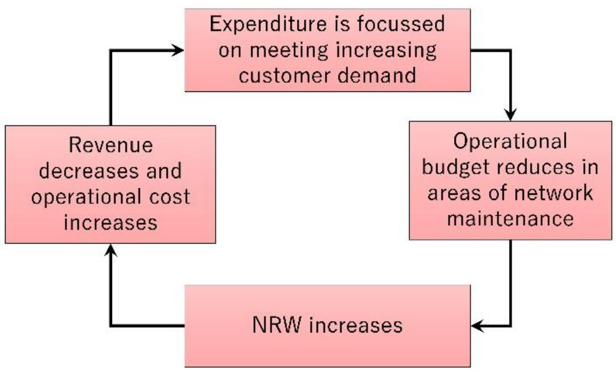

Figure 12.29: Vicious Circle of NRW

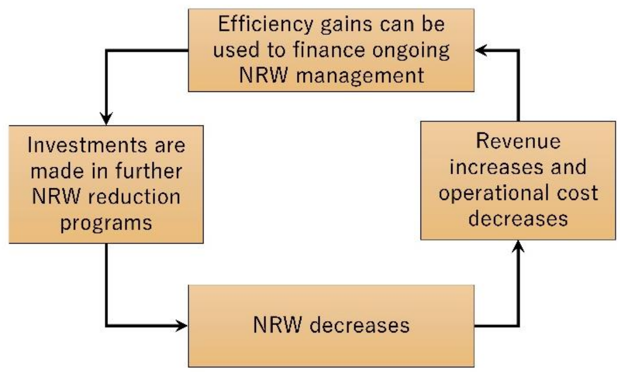

Figure 12.30: Virtuous Circle of NRW

The challenge for these utilities is to turn this vicious cycle (Figure 12.29) into a virtuous cycle (Figure 12.30), which will lead to low levels of NRW and therefore substantially improved efficiency. In most cases, many municipal organizations 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 operational zones. A city is divided into a number of operational zones (OZs) which are further divided into number of sub zones called as District Metered Areas (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 programs 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 program 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 Water Audit and Leakage Control of Part B of this Manual

Water Balance

Effective water management scheme aims at understanding the standard water balance and minimizing/avoiding non-revenue water. 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) non-revenue water.

Components of Water Balance: They are as below.

a) **Authorized Consumption:** It includes the volume of metered and/or un-metered water taken by registered customers. Authorized 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 Authorized Consumption:** It includes consumption of the consumers who are metered and billed and are producing revenue. It also includes billed unmetered consumption.
   ii. **Unbilled Authorized 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="8">System Input Volume</td><td rowspan="4">Authorized Consumption m3/year</td><td rowspan="2">Billed Authorized Consumption m3/year</td><td>Billed Metered Consumption (Including water exported)</td><td rowspan="2">Revenue Water m3/year</td></tr>
<tr><td>Billed Unmetered consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption m3/year</td><td>Unbilled Metered Consumption</td><td rowspan="6">Non-Revenue Water m3/year</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="4">Water Losses (NRW) m3/year</td><td rowspan="2">Commercial Losses m3/year</td><td>Unauthorized Consumption</td></tr>
<tr><td>Metering Inaccuracies and Data handling error</td></tr>
<tr><td rowspan="2">Physical Losses m3/year</td><td>Leakages on Transmission and Distribution Mains</td></tr>
<tr><td>Leakages and Overflows at Utility's Storage Tanks</td></tr>
</table>

			Leakage on Service Connections up to point of Customer metering	

b) **Water Losses:** Water losses comprise of the commercial losses and physical losses.

i. **Commercial Losses:** These include unauthorized 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 flow meter at entry point of each DMA, it becomes a tool for monitoring the NRW. NRW has two components, physical and commercial losses. Both of these components can be monitored. NRW in DMA is computed by the following equation,

DMA NRW = Total DMA Inflow- Total DMA Consumption (12.28)

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 meters 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 consumers 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 12.31.

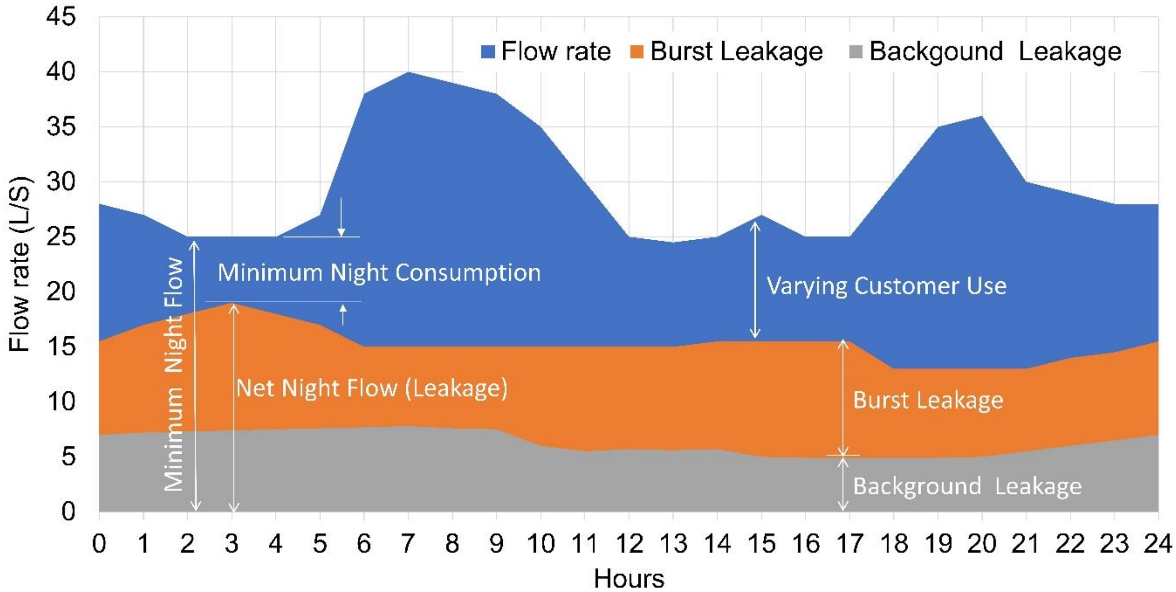

Figure 12.31: Flow pattern in a typical DMA

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 12.31. 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 can be computed which is as below,

$$NNF = MNF - LNF \quad (12.29)$$

12.15.2 Estimating Commercial Losses

Since, Total NRW = Physical Loss + Commercial loss, Commercial loss is computed by,

$$\text{Commercial losses} = NRW - NNF \quad (12.30)$$

12.15.3 Leak Repair Program

Bursts can be identified by the variation in minimum night flow over longer period, say 180 days. A typical such variation in a DMA is shown in Figure 12.32. These variation in night consumption can be observed and then can be identified and repaired.

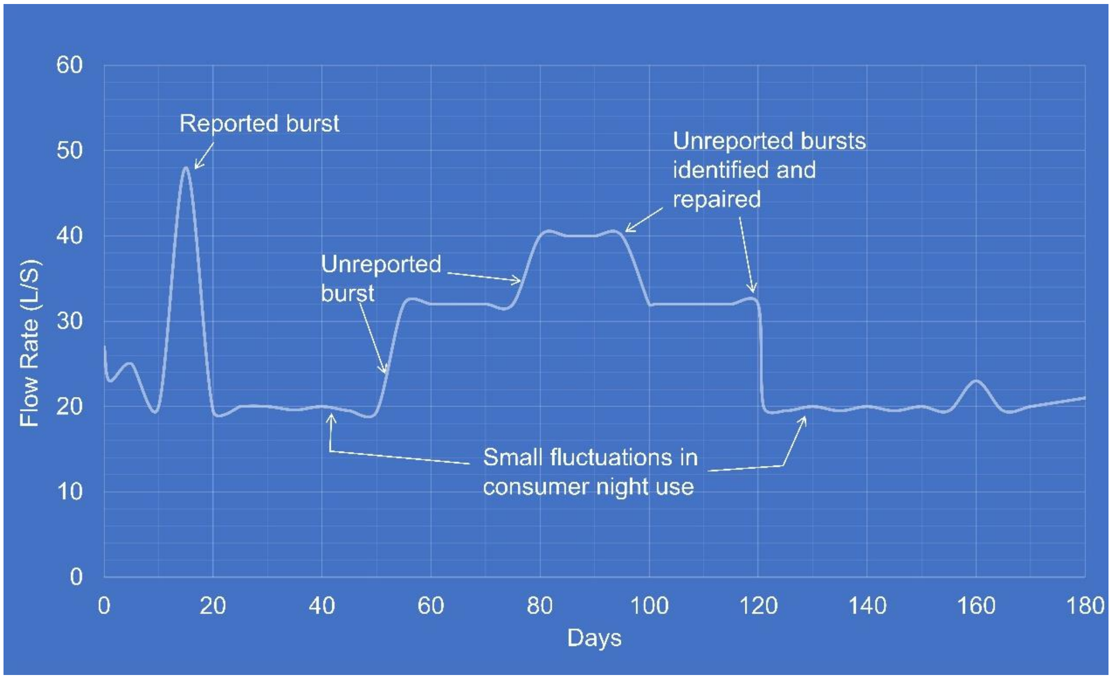

Figure 12.32: Fluctuation in minimum night flow over 180 days

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 shown as background leakages in Figure 12.31.

Unreported bursts can also be detected as shown in Figure 12.32. One of such unreported bursts is appearing on day 45, since it is not removed, the losses are continued and again another unreported burst occur on day 67. When both the unreported bursts are removed, NRW level is brought down as shown in graph.

12.15.4 SCADA Attached to DMA

DMA meter can be connected to the Supervisory Control and Data Acquisition (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 minimum night flow 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, Net Night Flow (NNF) should be recorded. As shown in Figure 12.33, 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, value of NRW again increases. Operator has to again bring NRW to its base level. If frequency of intervention increases rapidly, then the pipe replacement should be made.

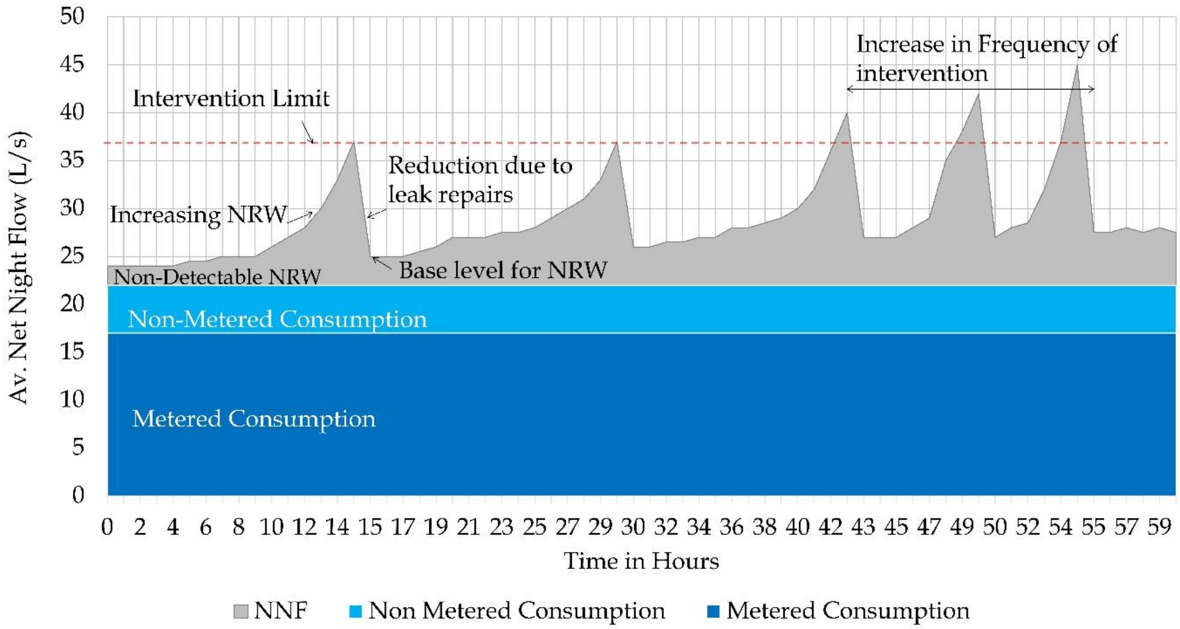

Figure 12.33: Increase of NRW and intervention with respect to time

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 12.35). 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 section, called as 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 minimum night flow (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 is approximate NRW value.

Illustrative Example: Network of DMA is shown in Figure 12.34. Using isolation valves, 9 segments are created which are shown in Figure 12.34.

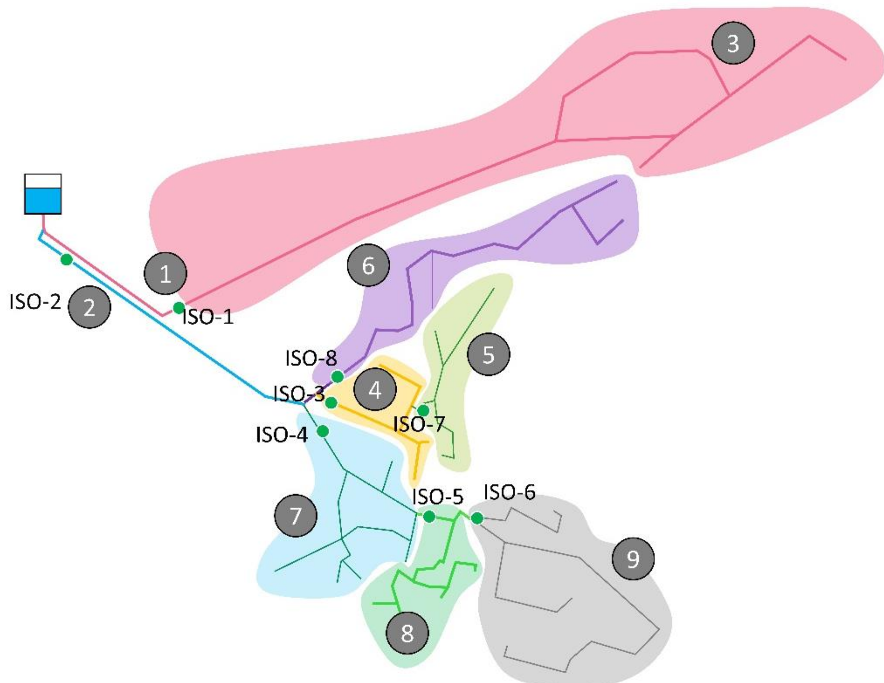

Figure 12.34: Nine segments (numbers shown in black circles) in DMA created by isolation valves (Mulay and Bhole 2021)

Test Procedure: The test started at 12 PM (00:00 hours) for determination of *Minimum Night Flow* (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 liters per minute) are noted.

Table 12.4: Sequence of valve operations and computation of NRW

S. No.	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 12.35. Also, MNF (NRW) in each segment is computed.

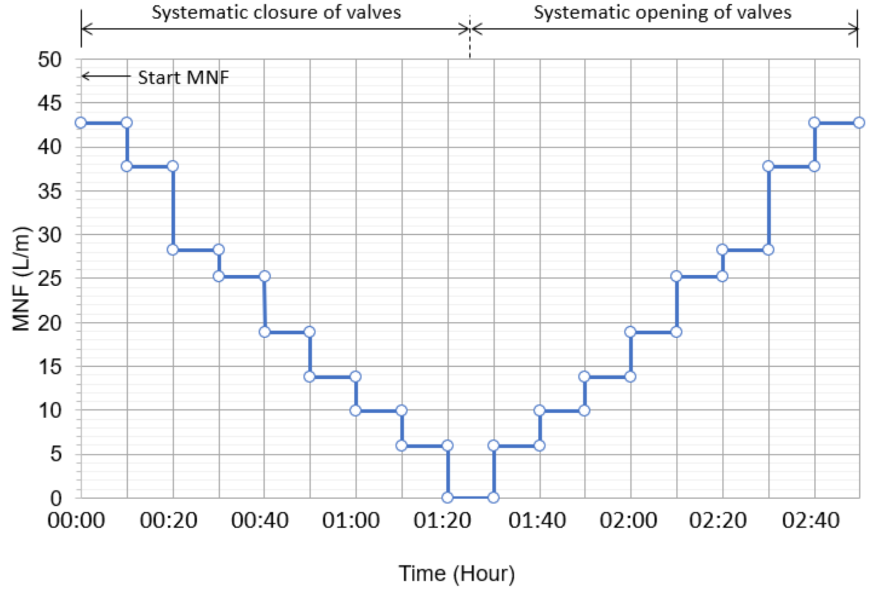

Figure 12.35: Values of MNF Vs. Time

Before conducting the STEP test, legitimate night flow (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 legitimate night flow was established and was observed as 4.9, say 5 Liters 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 Figure 12.35.

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 non-revenue water (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 & 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 and Gupta 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 minimizing 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

Water distribution system 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

Govt. of India and 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 24x7 continuous water supply. 24x7 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 Urban Local Bodies (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 Non-Revenue Water (%)	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 24x7 can be summarized as under:

- Planning and design
- Actual Conversion to 24x7
- Long-term operational stage

The above strategy is summarized 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 organizational, commercial, policy and budget are equally important. Summary of these strategic measures are shown in Figure 2.19 in Chapter 2.

All the above measures should be taken into consideration. If technical measures alone are taken but other measures are not taken, then the goal of conversion to 24x7 would not be achieved.

Activity Chart for Adoption of 24x7 Water Supply

Following Figure 12.36 shows the common activities which may be considered by ULBs /Water authority for adopting 24x7 water supply system.

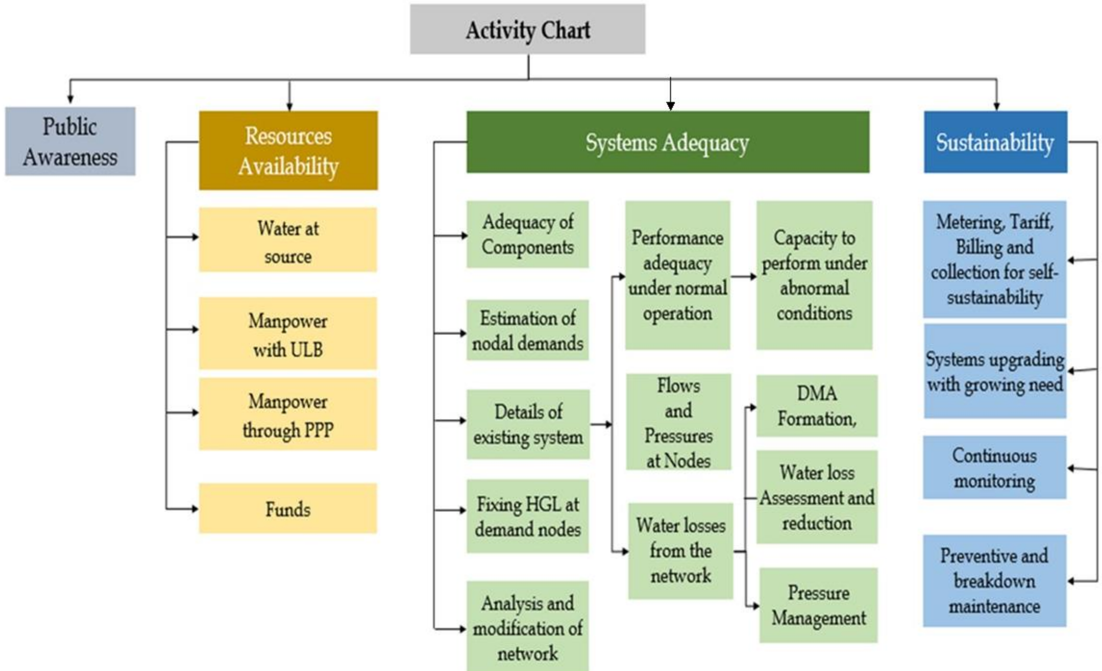

Figure 12.36: Activity Chart for Adoption of 24x7 Water Supply System

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Annexure 12.1: Capacity of Service Reservoir

Example1: Find the capacity of a newly proposed service reservoir serving operational zone having a demand of 2 MLD. The supply hours are 22 hours, and the supply shall shut during 21 and 22 hours. The hourly multiplying factors are shown in Figure 1. The initial water elevation is 100m and the maximum water elevation in the tank is 104m.

It is required to compute the capacity of the service reservoir.

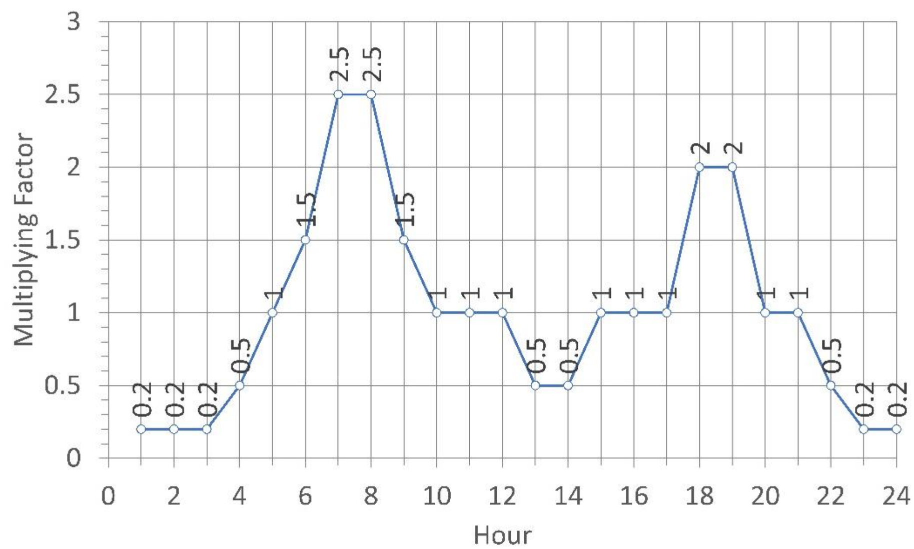

Figure 1: Multiplying factors

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 x 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.

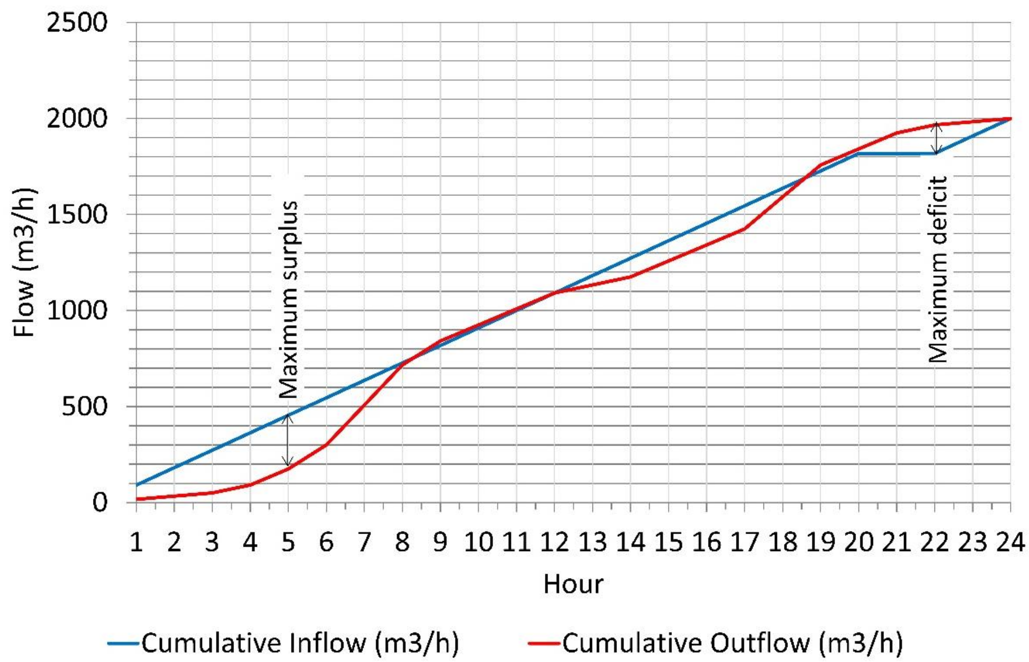

Figure 2: Graph of Cumulative Inflow and Cumulative Outflow with time

Capacity: For the assumed initial water elevation of 1.35m, it is observed that the values of cumulative inflow are more than those of the cumulative outflow till 8 hours and their difference is maximum at 5 hours (Figure 2). This means the surplus water is maximum at 5 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 8 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 24x7 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,

$$\text{Initial water level in the tank} = \frac{\text{Maximum deficit volume}}{\text{Area of cross section}} \qquad \text{(Eq. 2)}$$

In above case, maximum deficit = 148.5 (Col 7 of Table 1), hence,

$$\text{Initial water level in the tank} = \frac{148.5}{0.7854(12*12)} = 1.31, \text{Say } 1.35\ m$$. 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:

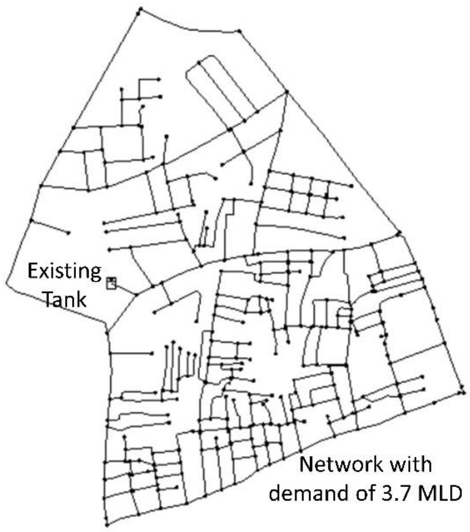

Figure 3: Existing ESR of 1 ML capacity and its operational zone

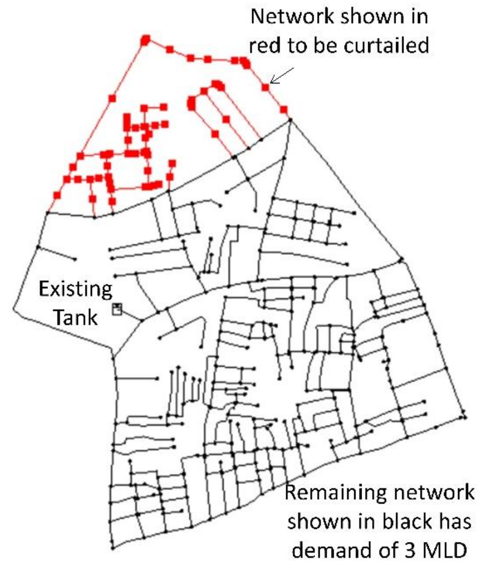

Figure 4: Curtailed network of the existing ESR of 1 ML capacity, with modified operational zone

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 operational zone 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

District Metering Areas (DMA)s are the building blocks of any 24x7 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.

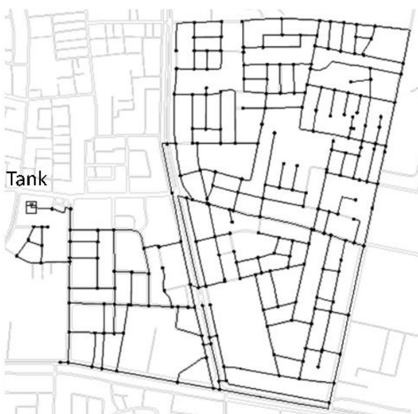

Figure 1: Before retrofitting

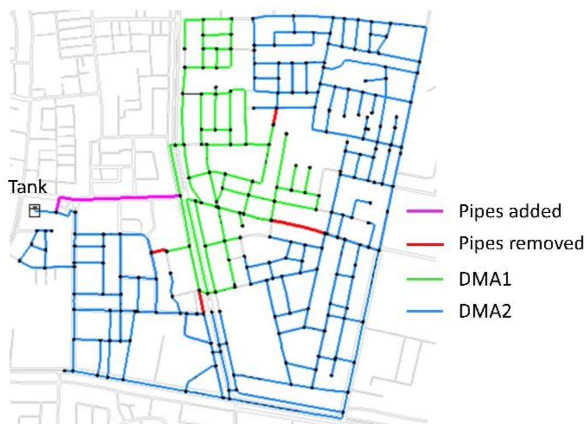

Figure 2: After retrofitting

One Operational zone is shown in Figure 1. It is required to divide this operational zone into 2 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 operational zone (OZ) and each OZ to receive water from only single reservoir. This means operational zone should be hydraulically discrete (isolated) from other operational zones. The process of doing this is illustrated in Figure 3.

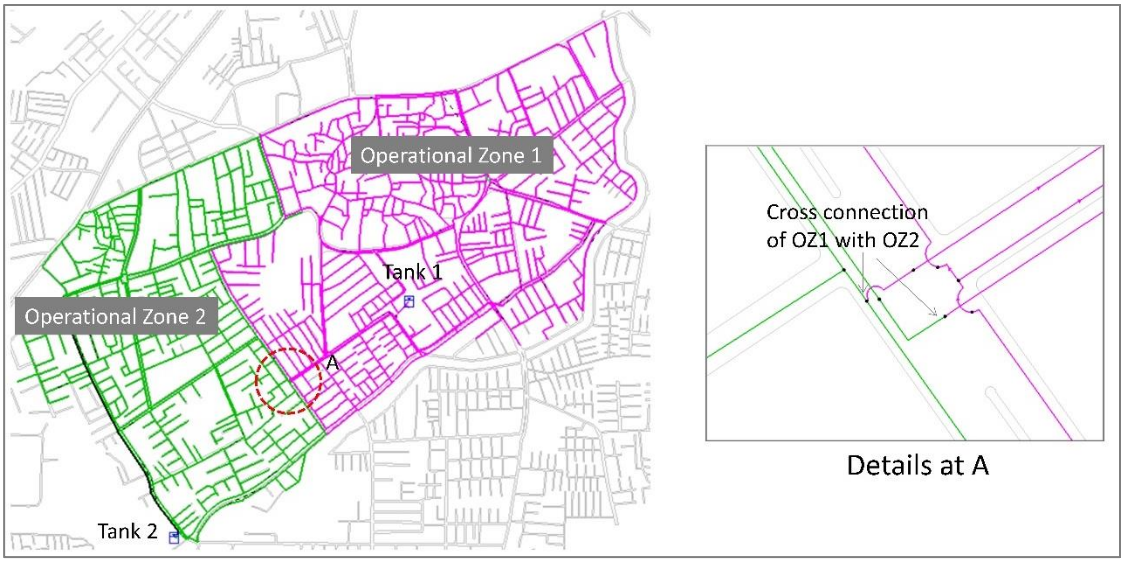

Figure 3: Cross connections between OZ1 and OZ2

The Operational Zones 1 and 2 with cross-connection between them are shown in Figure 3. These operational zones should be hydraulically discrete, and hence the cross connection should be removed.

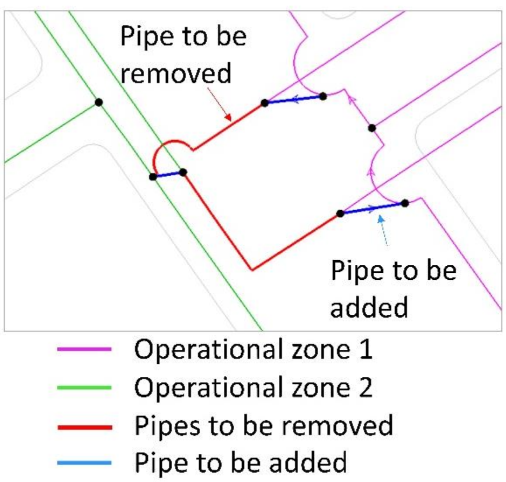

Figure 4: Before retrofitting OZs

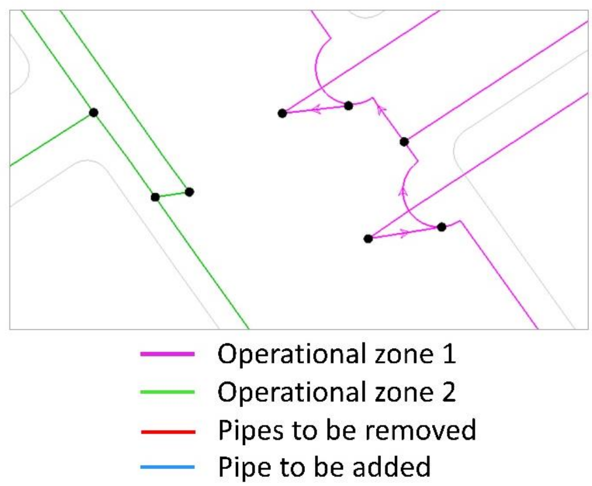

Figure 5: After retrofitting OZs

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 operational zones are selected depending on the ground elevations. Figure 6 shows such area wherein difference in elevations is more than 20m. 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 operational zone, 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

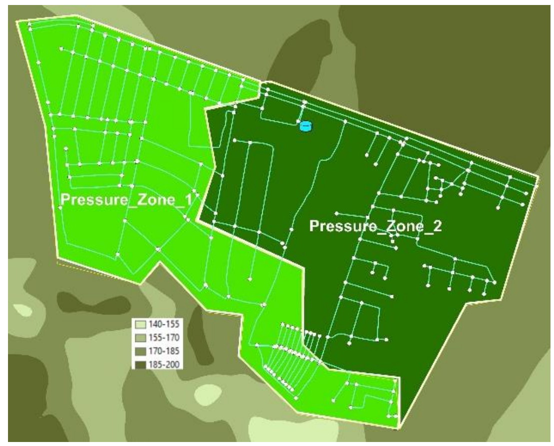

Figure 7: Marking existing pipe network in pressure zones

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.

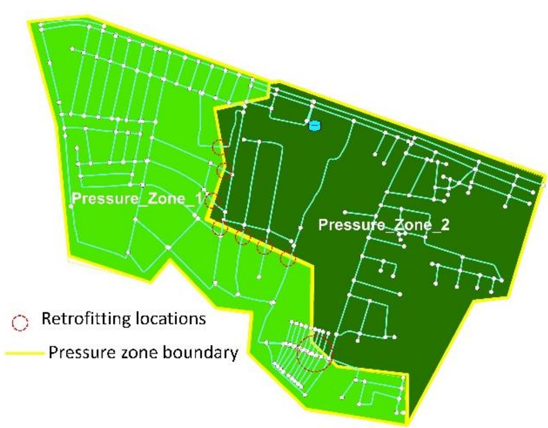

Figure 8: Retrofitting required to separate out pipe from pr. zones

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 Nodal Pressure

The objective is to identify the areas in operational zones in which the nodal pressures are less than the norm (Say, 21m) and then explain the procedure how to increase 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. On-line boosting on outlet of tank for entire OZ having nodal pressures < 21m
4. On-line boosting on branch line to area having nodal pressures < 21m

1. Separate out Part of OZ of Existing Tank: Consider one operational zone (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.

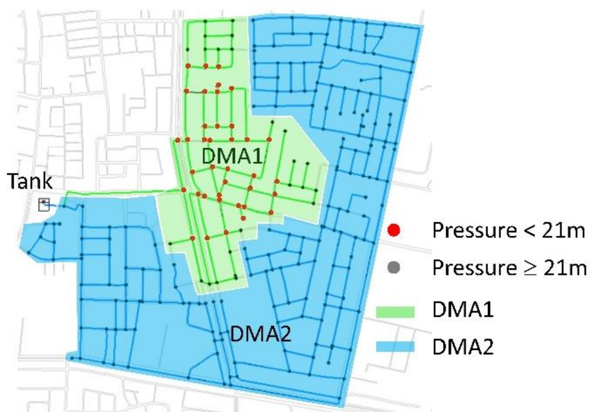

Figure 1: Nodes in DMA 1 have pressures < 21m

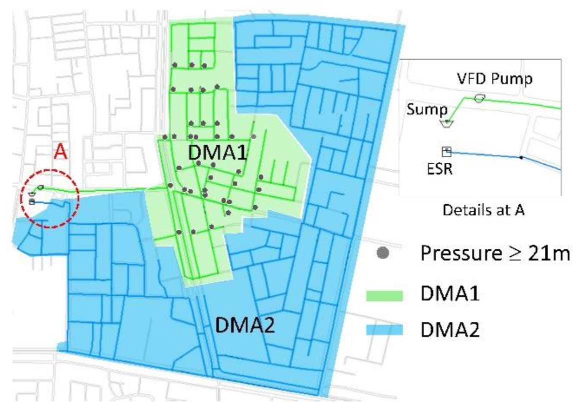

Figure 2: Direct pumping of DMA1 to increase pressures < 21m

One Elevated Service Reservoir (ESR) is shown in Figure 1 which supplies water by gravity to this operational zone. On running the hydraulic model, the nodal pressures are computed, some of the nodes in DMA1 have nodal pressures less than 21m which are shown in red colour in Figure 1.

Solution: Objective is to increase nodal pressure 21m. As nodes in DMA1 have nodal pressures less than 21m, 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 21m.

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 newly proposed ESR are proposed. The staging height of the newly proposed ESRs can be easily determined to get 21m residual pressure.

3. On-line 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 of Kotla ESR

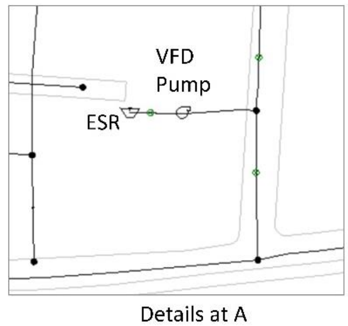

Figure 4: Pump installed on outlet of Kotla ESR

With small design head, Nodal pressures more than 21 m can be achieved.

4. On-line boosting on branch line to area having nodal pressures < 21m

On-line boosting on branch line to area (Figure 5) having nodal pressures < 21m

Figure 5: Area (shown in red) where pressures are less than 21m

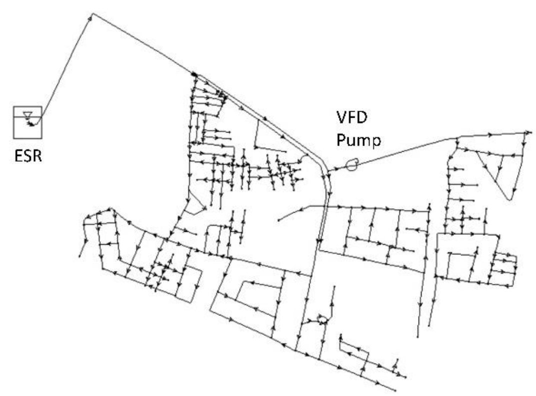

Figure 6: Network with VFD pump which is installed on branch pipeline which supplies water to area where pressures are less than 21m

In this case the VFD pump is installed on the branch (Figure 6) which supplies water to area where pressures are less than 21m. Now the nodal pressures are > 21m.

Annexure 12.4: Hydraulic Modelling Using VFD Pump

A case study of hydraulic modelling of one operational zone of the Ayodhya city is discussed here. One operational zone (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 elevated service reservoir 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.

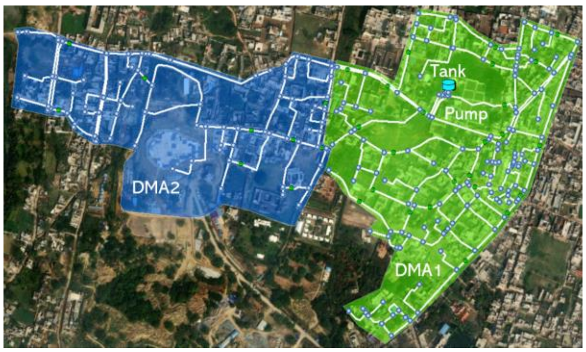

Figure 1: Network with DMAs

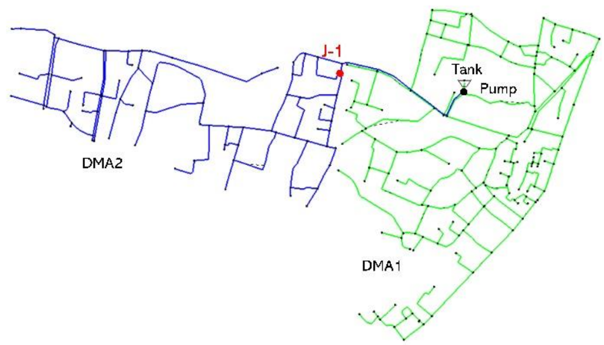

Figure 2: Network of one operational zone of Ayodhya

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.

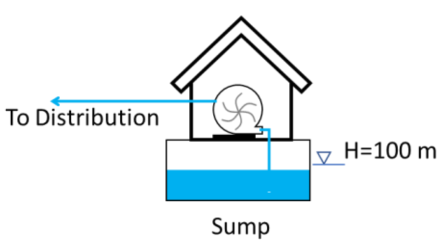

Figure 3: Option 1- Direct pumping from the sump

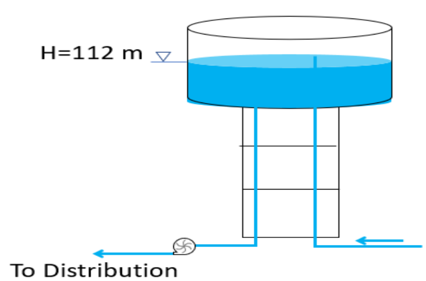

Figure 4: Option 2- Boosting by installing pump on the outlet of the tank

The required head for the pump to meet the residual pressure requirement of 21m 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, the 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 21m. For the 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 7 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 all non-peak hours.

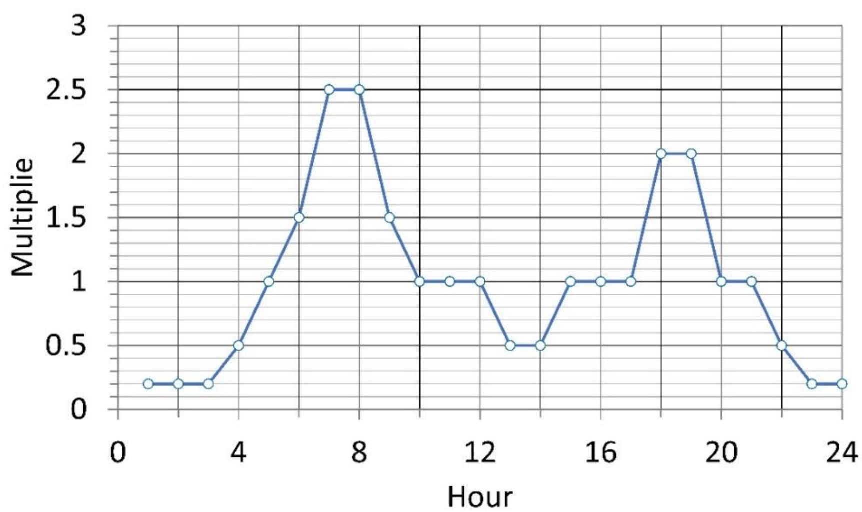

Figure 5: Peak factor

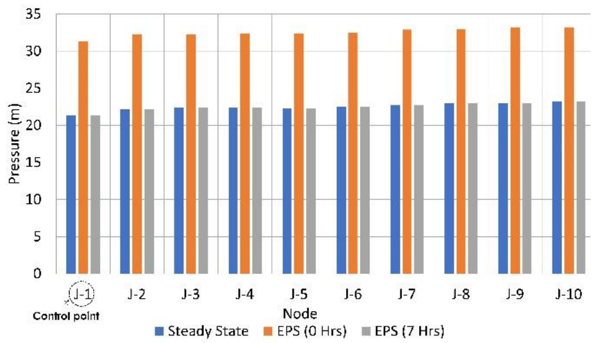

Figure 6: Option 1: Direct pumping from sump Steady State

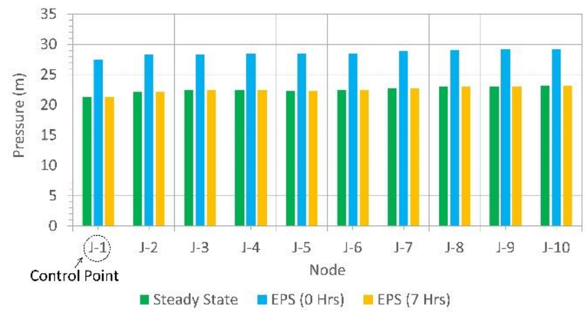

Figure 7: Option 2: Pumping by booster on outlet of tank.

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 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 21m at the critical node J-1 at all the time.

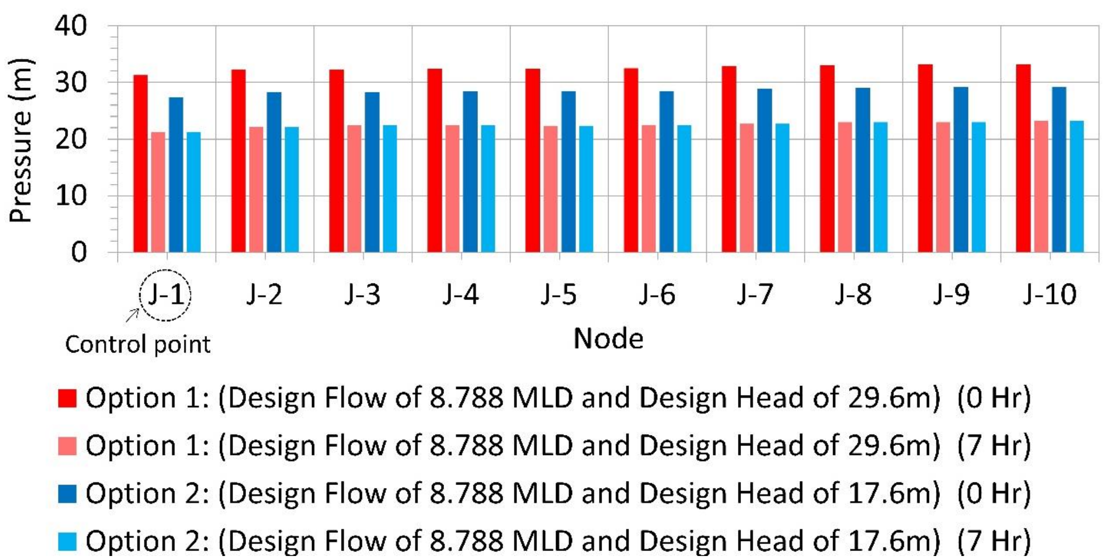

Figure 8: EPS results of pressure at nodes

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 (https://communities.bentley.com) 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 software calculates the values of relative speed factor of the pump for all the heads and flows for different points of time in a day. 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 the running the model is to compute the Relative Speed Factors at different point of times in a day which are required to maintain the 21m 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 elevation of 100m 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 elevation of 112m 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.

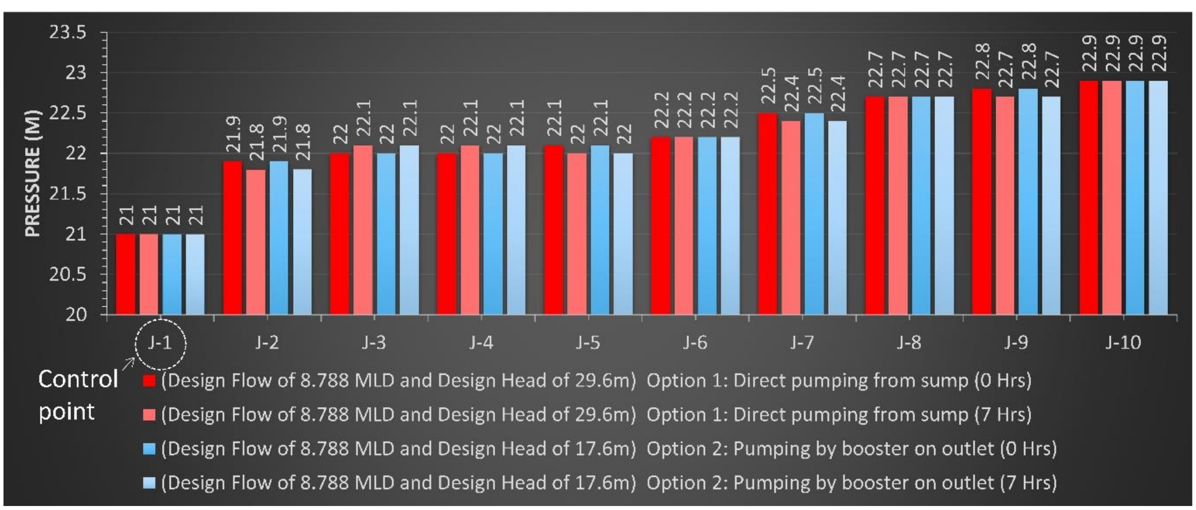

Figure 9: Pressure at nodes

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 21m in both Options 1 and 2 and for 0 and 7 hours. In fact, the same is observed at other non-peak hours also. 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 29.6m)	(Design Flow of 8.788 MLD and Design Head of 17.6m)
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
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 long run.

5. Conclusions:

The main advantage of 24x7 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 operational zone (Hanuman Garhi) of the Ayodhya city has been modelled in the one commercial software using both types of pumps. For maintaining the 21m 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' Catalog 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, if 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 pumps 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.

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2. https://communities.bentley.com
3. 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.
4. Walski et al. (2010), “Developing System Head Curves for Closed System,” Journal AWWA, September 2010

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Annexure 12.5: Gradient Method of Network Analysis

In the section 12.6.4 (d) gradient method for network analysis is described. 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 node 3, 4 and 5. Total number of pipes are six.

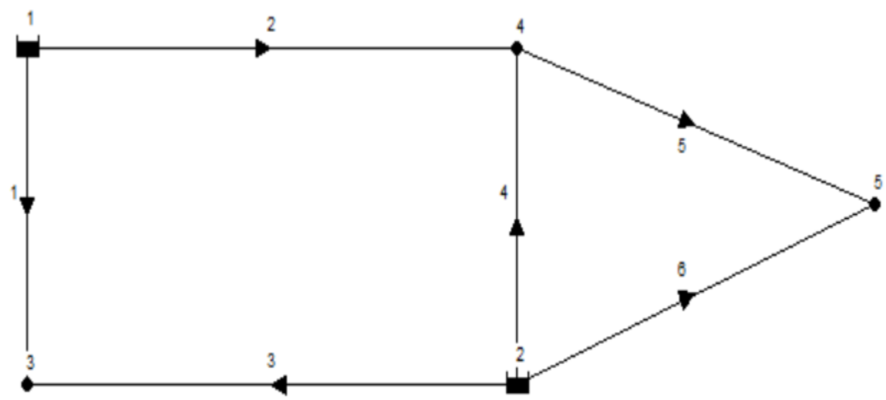

Figure 1 Two-source, three-demand-node looped network

The HGL at the source nodes 1 and 2 are: H_1=100m, and H_2=95m. The length of pipes 1 to 6 are: L_1= 200m, L_2=200m, L_3=200m, L_4=200m, L_5=300m, and L_6=300m. The pipe diameters for pipes 1 to 6 are: D_1=250mm, D_2=300mm, D_3=300mm, D_4=250mm, D_5=250mm, and D_6=250mm. The H-W constant for all the pipes is 100. The nodal demands at nodes 3 to 5 are q_3=0.3m^3/s, q_4=0.2m^3/s, and q_5=0.1m^3/s.

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

$$\mathbf{{}_{t+1}H = -A_{21}\,(NA_{11})^{-1}\,A_{12})^{-1}\,[A_{21}\,(NA_{11})^{-1}\,(A_{11}Q + A_{10}\,H_0) - (A_{21}Q - q_0)]}$$

$$\mathbf{{}_{t+1}Q = (I - (NA_{11})^{-1}\,A_{11})\,Q - (NA_{11})^{-1}\,(A_{12}\,{}_{t+1}H + A_{10}\,H_0)}$$

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 the 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 $\mathbf{{}_{t+1}Q}$ following step by step analysis, the following matrices are obtained.

$$\mathbf{A_{12\ t+1}\ H} = \begin{bmatrix} 182.7127 \\ 183.1827 \\ 182.7127 \\ 183.1827 \\ 249.3705 \\ 432.5532 \end{bmatrix}; \quad \mathbf{A_{12\ t+1}\ H + A_{10}\ H_0} = \begin{bmatrix} 82.71271 \\ 83.18271 \\ 87.71271 \\ 88.18271 \\ 249.3705 \\ 337.5532 \end{bmatrix}$$

$$\mathbf{(NA_{11})^{-1}\ (A_{12\ t+1}\ H + A_{10}\ H_0)} = \begin{bmatrix} 0.173357 \\ 0.423658 \\ 0.446729 \\ 0.184822 \\ 0.348436 \\ 0.47165 \end{bmatrix}$$

$$\mathbf{(NA_{11})^{-1}\ A_{11}} = \begin{bmatrix} 0.539957 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.539957 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.539957 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.539957 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.539957 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.539957 \end{bmatrix}$$

$$\mathbf{I - (NA_{11})^{-1}\ A_{11}} =$$

$$\begin{bmatrix} 0.460043 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.460043 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.460043 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.460043 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.460043 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.460043 \end{bmatrix}$$

$$\mathbf{(I - (NA_{11})^{-1}\ A_{11})\ Q} = \begin{bmatrix} 0.460043 \\ 0.460043 \\ 0.460043 \\ 0.460043 \\ 0.460043 \\ 0.460043 \end{bmatrix}$$

$$_{t+1}Q = (I - (NA_{11})^{-1} A_{11})\, Q - (NA_{11})^{-1} (A_{12}\, _{t+1}H + A_{10} H_0) = \begin{bmatrix} 0.286686 \\ 0.036386 \\ 0.013314 \\ 0.275222 \\ 0.111607 \\ -0.01161 \end{bmatrix}$$

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 5th iteration are same upto 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 down take 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 down-take 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 down-take 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 down-takes, as many as necessary, may be taken from, the loop and should be linked to one down-take for a zone or 4 storeys at a time and designed for the peak demand it has to serve. A pressure reducing valve shall be provided in the down-takes 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 5 to 8 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 down-take 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 7 storeys or 20m, 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 stand by 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 air tight vessel, cylindrical in shape and fabricated from mild steel plates minute’s requirements. The air-water ratio in the vessel.

b) Fire Storage

Multi-storeyed buildings above 25 m height have to 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 also maintain a reserve storage for firefighting purposes.

The firefighting pumps may be located in the basement to have a positive suction head and designed to deliver 2250 lpm with a terminal pressure of 3kg/cm2 at the top most floor so as to obtain from the hydrant 900 lpm discharge with a jet of about 6m.

On the firefighting rising main, hydrant tees of 60 mm maybe provided at every landing of each stair case. 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 automatically come into operation in the event from the generating set should be for all pumps including fire pumps, emergency lights, lifts and lights in stair cases and yards.