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MINISTRY OF HOUSING AND URBAN AFFAIRS
GOVERNMENT OF INDIA

स्वच्छ भारत
एक कदम स्वच्छता की ओर

MANUAL ON STORM WATER DRAINAGE SYSTEMS

VOLUME-I

PART A: ENGINEERING DESIGN

FIRST EDITION

CENTRAL PUBLIC HEALTH AND ENVIRONMENT ENGINEERING ORGANIZATION (CPHEEO)

http://mohua.gov.in | http://cpheeo.gov.in | https://swachbharaturban.gov.in

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CHAPTER 1: INTRODUCTION

1.1 General

Urbanization is taking place at brisk pace in India. In the first decade of 21st century, the number of towns increased from 5161 (2001) to 7935 (2011). Similarly, the urban agglomerations are not only increasing in number, it is also getting larger in population and sprawling. The number of cities with a population of 1 million or more has increased from 35 in year 2001 to 53 in year 2011. Majority of urban areas, be it large metropolis or small municipal town, severely lack with effective storm water drainage facilities. Unplanned development coupled with encroachment of existing natural drainage corridors, water ways etc. exacerbates the problem of urban drainage. In the quest for extreme development, important environmental benefits from natural functionaries like water ways/water bodies are often ignored, overlooked and compromised. This aspect along with recent trends in climate change is also causing the rise in incidences of acute water logging, urban flooding and related adverse economic and health impacts. Storm flows, if not regulated or routed to its convergence of safe disposal, may cause unprecedented degradation of urban infrastructure causing severe damage to life and property, depending on the degree of severity of storm event.

In recent years, frequency of urban flooding has increased and the issue is getting more pronounced day-by-day due to its enormous socio-environmental hazards leading to traffic snarls and disruption in urban life. Some of the notable cases of flooding which caused devastating impact on economic loss as well the loss of lives are urban flooding in Hyderabad (2001 and 2012), Delhi (2002, 2003, 2009, 2010 and 2011), Chennai (2004 and 2015), Mumbai (2005, 2008 and 2009), Kolkata (1978 and 2007), Surat (2006), Jamshedpur (2008), Guwahati (2010), Jaipur (2012), Jammu & Kashmir (2014) and recent devastation in Kerala (2018).

To protect the urban areas against flooding in phased manner, consistent with availability of resources, storm water drainage systems need to be planned and implemented in structured manner considering various aspects of design, operation & maintenance and economics. This essentially spells out the need for the formulation of this national manual containing aspects of planning, rainfall analysis, designing, detailed engineering of facilities, construction, as well as, operation & maintenance of urban storm water drainage systems including rehabilitation and refurbishing of prevailing under-performing drainage systems.

1.2 Status of Urban Drainage System in India

Urban storm water drainage systems have got priority in India only after drinking water supply and sewerage projects in majority of the cases. Due to fast pace of urbanization and migration of people from rural areas to urban areas in quest of livelihood and better education, there has been immense pressure on urban infrastructure, worsening the problem of urban drainage systems in India. Some of the important factors responsible for present status of poor urban drainage system in India are as under:

1. The natural drainage systems in most of the cities are in jeopardy and the problem of flooding is worsening with time due to non-availability of properly engineered storm water drainage infrastructures. The problems are exacerbated due to encroachment and rampant dumping of garbage & solid waste in the drains on one hand and absence of preventive maintenance on the other hand.

2. The megalopolises (mega cities) have a long history of municipal drainage perceptions since the British era. Most of the underground drainage facilities within core clusters of these mega cities are usually century old antiquated brick masonry conduits. The existing storm water collection network in these cities is mainly designed to serve as a combined system for sewage as well as storm water runoff. Augmentation and rehabilitation including separation of storm water from sewage in such facilities invite highest level of challenges for municipal engineers and financial resources

3. The coverage of storm water drainage network stands about 20% of road network and its allied catchments as per the report on Indian Urban Infrastructure and Services (March 2011), published by the erstwhile Ministry of Urban Development (MoUD) which is too inadequate to cater the storm water disposal in the present city scenario.

1.3 Causes of urban flooding

Some of the major causes leading to frequent flooding even in light rainfall in urban areas of India are as under:

1. The average annual rainfall in many important cities in the country receives high rainfall during four months of monsoon. The cities like Mumbai receive annual average rainfall of order of 2,932 mm. High intensity rainfall in such cities is responsible for frequent flooding.

2. Storm drainage systems are currently designed in India for rainfall intensities of once in one year to once in two years return periods. In case, rainfall records are not available, rainfall intensity is usually adopted in the range of 12 mm/hr – 20 mm/hr. Therefore, ‘Accommodation & Transportation’ capacities of such hydraulically configured drainage facilities are easily overwhelmed, whenever rain storms of higher frequencies are experienced.

3. Unplanned urbanization causes considerable increase in impervious areas, thereby, leading to enhanced surface runoff and frequent flooding.

4. The problem of disposal of storm runoff is compounded in the cities having flat terrain, tidal fluctuations in coastal areas and blockage of streams / drains due to landslides in hilly areas.

5. Global climate change resulting in changed weather pattern and increased occurrence of high intensity rainfall events further aggravate the risk of flooding in towns and cities.

6. The absence of systematic approach to formulate and implement a holistic storm water drainage scheme within specified planning horizon has turned urban areas and cities so vulnerable now that even rain of light and medium intensity causes urban flooding.

7. The problem of illegal disposal of Construction and Demolition waste / municipal solid waste coupled with poor maintenance of existing drainage system often obstructs the storm runoff causing localized flooding in the areas.

1.4 Need for Storm Water Drainage Manual

In view of the existing status of drainage systems in urban areas, that causes frequent flooding leading to loss of property and life, it necessitates looking into the problem more closely and coming out with planning, designing, implementation and operation & maintenance guidelines to overcome the problems in urban areas.

It is worth mentioning that there is no dedicated reference document/ manual available on storm water drainage systems in India like the ones on Water Supply & Treatment Systems, 1999; Sewerage and Sewage Treatment Systems, 2013 and Municipal Solid Waste Management, 2016, all published by the Ministry of Housing and Urban Affairs, Govt. of India. Majority of the countries across the globe like Singapore, Hong Kong, Australia, United States of America, Malaysia, and many others have their own manual on storm water drainage systems. The existing “Manual on Sewerage and Sewage Treatment Systems, 2013”, published by Ministry of Housing and Urban Affairs, Government of India, incorporates very few aspects of urban storm water management under its Section 3.9, but the same is grossly inadequate to address the issues related to storm water drainage.

Therefore, a comprehensive and exclusive manual on storm water drainage systems has become imperative to guide the public health engineers / municipal engineers / city planners and consultants across the country. Accordingly, this national manual is formulated to provide necessary guidance towards all aspects of sustainable design, planning and management of storm water drainage systems of towns and cities of the country.

1.5 Scope of Manual

This Manual is a guide book for hydrologic & hydraulic design of storm water drainage systems and includes, inter alia, planning, designing, detailed engineering and operation and maintenance of various components of urban storm water drainage systems. The appurtenant socio-environmental aspects, are also an embedded component of the scope. The manual is contained in three distinct Parts over two volumes. Volume 1 contains Part A: Engineering Design whereas Volume 2 contains Part B: Operation, Maintenance and Part C: Management. A brief outline of these volumes is as follows:

Volume 1 - Part A: Engineering Design

The Volume 1 of Manual contains the planning and Engineering Design of various elements of storm water drainage systems and is organized in the following Chapters:

- **Chapter 1** deals with the issues of urban drainage system in India and requirement of an exclusive Manual on Storm Water drainage Systems.

- **Chapter 2** covers the planning aspects of urban storm water drainage projects, related Investigation, data collection, survey inputs, and environmental as well as financial aspects.

- **Chapter 3** contains rainfall analysis and development of IDF curves using empirical and probability methods.

- **Chapter 4** deals with runoff estimation resulting from urban catchments by various methods viz. rational method, time area method unit hydrograph method and rainfall runoff simulation method.

- **Chapter 5** covers hydraulic design of storm water drains and conduits with applicable flow equations.

- **Chapter 6** covers additional design considerations for special areas to be considered while designing storm water drains in Hilly and coastal areas.

- **Chapter 7** deals with structural design of storm water drains covering process design of underground rigid and flexible conduits for carrying storm water.

- **Chapter 8** covers handling of storm water through pumping and deals with planning and design of pumping station for pumping of storm water.

- **Chapter 9** covers planning and design of rain water harvesting to be used for multiple purposes like artificial recharge of ground water and attenuation of flash floods.

- **Chapter 10** elaborates the methodology for using existing drains to allow excess runoff to percolate in the ground water. It also deals with various emerging practices adopted across the world for storm water drainage design such as Water Sensitive Urban Design (WSUD), Low Impact Development (LID) and Sustainable Urban Drainage System (SUDS). Best Management Practices (BMPs) of storm water drainage system are also discussed briefly.

- **Chapter 11** covers construction aspects of storm water drains, conduits and its appurtenant structures.

Volume 2: Operation, Maintenance and Management

The Volume 2 of the Manual is divided into two parts as briefly described below:

Part B: Operation and Maintenance, deals with operation & maintenance, inspection and replacement & rehabilitation of storm water drains and conduits.

Part C: Management, deals with public awareness, capacity building, institutional arrangement and financial sustainability.

1.6 Use of Manual

This manual has been prepared for the purpose of assisting engineers, planners and designers working in government institutions, urban local bodies, industrial and business concerns, consultancy services, etc., in planning and design of urban storm water drainage system in the country.

The aim of the manual is to provide details of essential technical and engineering aspects considered during planning, design and management of urban storm water drainage system and to provide details of appropriate design and computational procedures.

CHAPTER 2: PROJECT PLANNING AND INVESTIGATION

2.1 General

The project planning and investigation is backbone of project development and lays the foundation for its successful implementation and economical operation and maintenance. This chapter covers various aspects of project planning and investigation, data collection, survey, design considerations, environmental considerations, permission and clearances, provision for O & M, financial sustainability etc. A checklist for preparation of DPR is also developed for objectively guiding DPR preparation and its vetting by the concerned authorities.

2.2 Objectives of Planning & Investigation

The objective of planning and investigation is to provide detailed information about the type and topography of the land, details of existing drains, land use pattern, networks of roads, culverts and railway lines etc. which required due consideration in planning / augmentation of drainage system in the city. The following aspects need to be considered while planning & investigating for development of a drainage system:

i. Identification and marking of probable drainage zones, direction of gradients and selection of disposal points
ii. Preparation of topographical layout of collection and conveyance.
iii. Identification of locations for pumping stations
iv. Strategy for rainwater storage and its recharge to ground water
v. Strategy for prevention of solid waste and C & D waste into storm water ways.
vi. Strategy for arresting pollutants with urban runoff from entering into water bodies
vii. Conserving the aesthetic, public safety and other social concerns of recreational open space and landscape to preserve the ecological nature of water ways;
viii. Identification of existing storm water drains / drainage corridors including age-old drainage conduits for rehabilitation;
ix. Non-structural measures should be studied and components designed accordingly to provide relief during occurrence of disasters due to flooding.
x. Frame a Road Map for Urban Storm Water Best Management Practices (BMP).

xi. Preparation of strategy for protection of urban areas from flooding

xii. Strategy for sustainable operation & maintenance of storm water systems

xiii. Holistic approach to local area planning including aspects of sustainability, consistency and responsive to community values.

2.3 Data Collection, Survey and Investigation

Before start of field survey, sufficient desk work should be carried out using the existing details and that should be corroborated by field visits and discussions with local community and municipal officials. This iterative process should be repeated to prepare a comprehensive workable plan. The data / information to be collected and the elements to be surveyed for preparation of project plan are given below:

2.3.1 Data Collection

The data collection shall comprise of the following but not limited to:

i. Physical Characteristics

- Topographical details including slope of catchment and outfall point ;
- Identification of existing and expected future land uses
- Details of Bridges, culverts, railway crossings etc.
- Areas of urban forest, wetlands, marshy lands, flood plains, water bodies etc.
- Data on inflows from contiguous upper regions;
- Soil characteristics including its permeability
- Ground water table and its seasonal variations,
- Potential of use of storm water in project area or adjoining area;
- Identification of storm drainage related problems within urban areas that may warrant further detailed investigations and planning such as:
    - Littering, garbage, domestic wastes etc.
    - Solid waste / C & D waste points nearer to the drainage system
    - Nearby dump site status,
    - Natural pollution, such as leaves, etc.
    - Chemical pollution, such as detergents, oil or fertilizers

ii. Rainfall Characteristics

- Rainfall data for last 30 years or more depending on availability from digitized / Automatic Rain gauge station needs to be obtained / collected in the specified format.
- Rainfall data collection comprising of annual average daily and monthly rainfall and no of rainy days
- Data on historical flood events

iii. Waterway Characteristics

- Capacity of water receiving body and its HFL and other relevant details
- Physical condition and characteristics of the existing (size, slope and material) storm water conveyance system;
- Existing natural, as well as, engineered drainage channels;
- Details of existing water bodies
- Location of existing and prospective rain water harvesting structures;
- Water quality & quantity in existing storm water conveyance systems / natural drains and in receiving water bodies under wet and dry conditions;
- Tidal influence on receiving water bodies for the catchment;

iv. Collection of topographical survey details / maps

Following documents/maps are needed to be collected for proper planning:

- Survey of India topographical maps (1:50,000) of the planning area for comprehension of topography, water courses and other physical features like major roads, railway lines, location and levels on bench marks etc.;
- Details of bench marks established by Survey of India in the planning area or its neighborhood,
- Existing aerial survey of the planning area;
- Digital data / satellite data;
- Local planning area maps and scheme maps of various scales prepared by various agencies such as Department of Town and Country Planning, Water & Sewerage Boards, Municipal Corporations etc. for comprehension of water courses, irrigation channels, storm water drains, tanks, temple, ponds etc;

- Reconnaissance survey for verification and updation of complete inventory of drainage system of the planning area consisting of water courses, irrigation channels, storm water drains, tanks, temple ponds etc.;
- Location of underground electric cables, telephone lines, water supply and sewer lines etc.
- Water shed maps including topographic features, water shed boundaries, existing drainage patterns and ground cover;

2.3.2 Survey and Investigation

After analyzing the collected data including the existing survey maps and existing drainage details, broad alignment for drainage network should be firmed up and survey should be commenced to collect the requisite data / field details for preparation of alignment of drains / maps with suitable ground levels.

For carrying out the survey, latest survey instruments like Total Station Survey / Drone/ aerial survey techniques etc. should be used. Based on survey, the coordinates and levels of various important locations / bench marks should be collected. Further, field survey for the project should include overall infrastructure mapping, strip survey and site survey. During the topographical survey, traversing should be done along the centre line of the corridor. Longitudinal cross-sections should be taken at intervals as required for clartity. Also, the final data should be converted in Environment System Research Institute (ESRI) (Shape file) format with its defining projection and survey collected attributes in the requisite database format.

The layout plan should be prepared from the GIS database after integration with selected computer model, adopting the Arc GIS operating environment. Layers and attributes to be shown in the map should be flexible to control and appropriate information for different requirements.

Based on above survey, following plans should be prepared:

- Topographical maps (1:1000) bringing out existing storm water drainage system, crossing of main watercourses e.g. rivers, irrigation channels and drains, tanks, ponds, roads, railway lines, built up areas, open fields and play grounds, flood prone areas etc.;
- Contour maps;
- Demarcation of urban catchment in sectors, zones and sub zones in order to plan layout of Primary, Secondary & Tertiary drains

- Alignment of watercourses showing locations of temporary/ permanent structures within 15 m on either side of the bank location of electric cables, telephone lines, water supply and sewer lines in the vicinity of the drains.
- Storm water drains with longitudinal section at 50 m interval and cross section at every 100 m interval or the change of cross section.
- Water harvesting structures, Water detention tanks, Pumping points, water usage points, parks, disposal point should also be shown on the map.
- Details in and around the drain for recharge should also be identified particularly at the places along stretch of drain where soil strata / log is changing indicating Type of soil, Permeability, Ground Water Table, Rock strata
- Identification of Vulnerable silting / landslide points, Low lying points Coastal area problem, Hilly area features / vulnerable stretches
- The above details collected can be used for planning of drainage system including its integration with existing drains and rehabilitation of other existing drains.

2.4 Other Considerations

2.4.1 Permissions and Clearances

The necessary permissions and clearances may be obtained in advance along the drainage alignment for the smooth implementation of project. Further, the permission for removal /relocation / or diversion of existing services should be taken up with concerned department at an adequate earlier stage within the ambit of project planning and implementation procedure because the process is normally lengthy. Town planners and engineers, therefore, shall start the process to obtain necessary government sanction at very early stage to avoid delay in project implementation.

2.4.2 Environmental Consideration

This pertains to the aspects to be considered in relation to the environment such as aesthetics, landscape, ground water recharge, etc

i. Environmental Assessment

Environmental impact assessment should be carried out in accordance with the procedures prescribed by Government of India under Environment (Protection) Rule 2006 and 2009.

ii. Aesthetics/Landscape

Urban Drainage infrastructure shall be so planned and designed that the same should holistically blend with surrounding environment. Aesthetic aspects should be stressed in structural designing and landscaping in order to create a symmetrical and perspective vision with spatial environmental back drop.

iii. Surface Water

Considerable quantities of trash and other debris are washed through storm drainage system into receiving bodies of water resulting as a primary impact in creation of an aesthetic eyesore in waterways causing reduction in recreational value, whereas, in smaller streams debris may generate blockage of the channel which may result in localized flooding and erosion. This shall be meticulously studied and remedial measures need be proposed.

iv. Ground Water

Increased urbanisation has resulted in the increase in percentage imperviousness and in turn precluding the natural infiltration process of storm runoff. Such phenomenon reduces ground water recharge rate and consequently lowering the ground water table. This aspect shall be taken into consideration while designing recharge structures and suitable treatment measures to be proposed if such recharge is contemplated to be carried out.

v. Coastal Water

Sediments, silts, debris etc. discharged through storm drainage system into coastal waters and recreational sea beaches may cause physical damage including degradation of water quality and smothering benthos. Nutrients such as nitrogen and phosphorus in excess in storm water may cause eutrophication resulting in excessive algal growth. This should be adequately taken care of.

2.5 Hydraulic Design of Storm Water Drainage Systems

Using the data collected above and topographical survey carried out the route of drains should be marked on the map along with ground levels and showing existing infrastructure including various other salient features as mentioned above. Also, the rainfall data should be collected and analyzed as mentioned in Chapter 3, and runoff estimation to be carried out in different stretches of drain alignment. Using this runoff data, the storm water drains should be designed following the aspects of hydraulic design as mentioned in Chapters 5, 6 & 7.

However, it may be mentioned that internal drainage of urban catchments may not be designed for peak flow for rare storm events such as 1 in 25 or 50 years or so but it is necessary to provide sufficient protection against excessively frequent flooding of the drainage area. The Design Return period are presented in Chapter 4.

There shall be obviously considerable flooding when the precipitation exceeds the 'Design Return Storm'. However, such flooding may have to be accepted in spite of once in a while inconvenience considering its occasional utilization in few instances in a year and the nature of cost intensive projects and its feasibility on ground due to various other utilities available along the road. However, in such situations, the preparatory measures to deal with such scenarios as specified by 'National Disaster Management Guidelines, published in September, 2010' (Chapter 3 and related sections)

2.5.1 Inlet locations

The storm water inlets (Catch pits / Catch-Basins) are mainly provided to accommodate the storm water from paved surfaces (Kerb & channels for large metropolis), parks, open space areas and transfer it to sub surface drains for conveyance to the ultimate 'receiving body'. Even where open drainage system is used, the inlets are connected to open drains by means of interconnection pipes. The inlets need to be hydraulically designed and suitably spaced. The detailed norms for design are in Chapter 5.

2.5.2 Manholes (MHs) and its locations

Manholes (MHs) in the sub-surface drainage system are provided at following locations:

- Major change in flow quantum due to addition of flows (junctions);
- Bends because of change in direction of alignment;
- Large drops in inverts because of topographical configuration;

The detailed norms for design are in Chapter 10.

2.5.3 Pumping of storm runoff

Storm runoff follows the gradient of the terrain in the drainage catchment. In many low lying stretches with flatter slope as well as near coastal areas, pumping arrangement has become necessary for efficient functioning of storm water drainage systems. While designing pumping system, the following basic aspects should be considered:

In the case of permanent pumping stations the following need to be considered

- Identification of pumping points
- Details of space availability
- Distance / route of rising main alignment
- Estimation of design runoff at pumping station
- Capacity of the wet well i.e. the detention time in minutes pertaining to the peak of the Routed Hydrograph at the Drainage Pumping Node;
- Additional storage capacity if required;
- Number of pumps including standby and operating point (Q vs H) of pumps, determined from synchronization of pump characteristics (single or in parallel) & system head curve and authenticated through NPSH, as well as, discharge vs power input curves and other typical elements related to pumping system;
- Electric motors or fuel engine driven pumps;
- Operation and maintenance requirement;
- Generator sets of appropriate capacity. In the case of diesel based transit pumping systems, the following need to be considered.

The detailed norms relating to pumping are mentioned in Chapter 8.

2.5.4 Outfall Structures

Location of outfall point should be selected considering the level of the surface water of receiving water bodies such as low water level, high water level and normal water level. Care should be taken that the outfall level should be adequately high above the High Flood Level. Wherever it is not feasible due to level of terrain adequate protection mechanism should be provided to check back flow of water in the outfall drain. Cascading and apron structure if necessary may be incorporated in the Outfall Structure System. The accessible location of out fall structures should be clearly shown on the plan. The detailed norms for design are mentioned in Chapter 6.

2.5.5 Natural Streams/ River

In cases of probable flooding of the catchment due to natural streams/rivers, flood protection measures should be employed without reducing the natural waterway.

2.5.6 Augmentation and Rehabilitation of Existing Drainage system

Existing drainage facilities need to be examined with respect to shape, size, material, invert information, out fall location(s), age, condition etc., consistent with volume of storm water flow and suitably augmented / rehabilitated to convey the designed runoff. Details of water logging incidences/ complaints from municipality/ police may also be obtained and incorporated.

2.6 Financing

Project implementation involves Capital as well as O & M cost.

Capital cost includes all initial costs such as civil construction, cost of drains appurtenances, pumping machineries installation and erection costs, opportunity cost (land cost in case of government land), engineering design and supervision charges, interest charges on loan if taken during construction period.

Financial viability of any project is as important as its technical viability and it can also be said that operating cost is more important than the capital cost to ensure sustainability of the project. The benefits from storm drainage project may not be quantifiable in cash inflow terms but its social, health and other benefits can be assessed more than revenue accrual.

For efficient functioning of storm water drainage system proper operation & maintenance is essential which is possible only when O & M funds are available to take up maintenance activities. The fund may be earmarked from drainage activities from municipal budget and may be collected in the form of storm water drainage cess based on area of premises and level of construction done.

Annual Operating costs after the project is commissioned shall include the summation of the direct operating cost and fixed costs like amortization and interest on capital borrowings, direct operation and maintenance costs on the following:

- Staff
- Chemicals (if any)
- Fuel and electricity
- Transport
- Maintenance and repair
- Insurance
- Overheads etc.

  On the other hand the benefits arrived from such social engineering projects are multifarious in terms of:
- Direct revenue earning from the beneficiaries through development and betterment taxes with multilevel taxation putting minimum burden to the economically weaker section of the community;
- Direct financial gains attributable to: not incurring loss of business /properties individual, as well as, government because of the project;

- The indirect benefits in terms of improvement of general public health which can be termed as socio-environmental benefits;

If one can assess and quantify the summation of all such benefits, the Benefit/ Cost Ratio (B/C) for such social engineering projects are always expected to be more than unity.

2.7 Operation & Maintenance

For any system to operate in a proper and efficient manner, the key is its appropriate and planned regular and preventive operation and maintenance. These aspects of O & M have been addressed in Part B of the manual.

2.8 Citizen Awareness

Citizen awareness is the growing recognized mechanism to dissuade people from habit of indiscriminate littering and dumping of debris and solid waste either on open ground or nearby rivers/ streams/lakes/drains. This is dealt in detail in Part C of this manual.

2.9 Institutional Arrangement and Capacity Building

Creation of storm water drain infrastructure is one aspect but its periodic maintenance is the key to provide the desired level of services on a sustainable basis. An efficient organization is very important for planning, design, and sustainable operation and maintenance of SWD infrastructure. Therefore, measures must be taken for institutional strengthening and internal capacity building so that the efforts made can be sustained over a period of time and the system put in place can be well managed. Institutional strengthening can be done by adequately decentralizing the administration, delegating adequate powers at the decentralized level, inducting professionals into the administration and providing adequate training to the existing staff. These are dealt in detail in Part C of this Manual.

2.10 Service Level Benchmark

While planning a project, efforts should be made to perform as per Service Level Benchmark notified by Ministry of Housing and Urban Affairs, (MoHUA) Govt. of India as shown in Table 2.1 below:

Table 2.1: Performance Indicator

Indicator	Value
Coverage of storm water drainage network	100 %
Aggregate number of incidents of water logging reported in a year	0 per year

2.11 Checklist for DPR preparation

A checklist has been prepared and placed in Appendix 2.1 which can be referred to by the users of this manual towards preparation / scrutiny of DPRs of storm water drainage.

CHAPTER – 3: RAINFALL ANALYSIS

3.1 General

In storm water drainage system design, estimation of runoff from the tributary catchment reaching various inlets of drain is important. This can be estimated if Intensity Duration Frequency (IDF) curves are available. The IDF curve is drawn based on rainfall data analysis of the project area obtained from the daily rainfall charts of Self-recording Rain Gauge (SRRG) stations of Indian Meteorological Department (IMD).

In this Chapter, the rainfall data obtained from SRRG station of IMD has been analysed and the procedure for construction of IDF curve using Empirical method is explained. Once IDF curve for required return period are constructed, the same can be used for estimation of runoff using rational method. Probabilistic methods for constructing IDF curves have also been explained in brief.

3.2 Rainfall

Rainfall is a form of precipitation. The term precipitation is a generic term used to denote all forms of precipitation that reaches surface of the earth from the atmosphere such as rainfall, snowfall, frost, hail, sleet, drizzle, glaze and dew. Rainfall (water drops of size 0.5 mm – 6 mm) is the major form of precipitation that causes stream flow as well as flood flow in rivers. Variation in magnitude and duration of rainfall in different parts of the country leads to potential of flooding of urban areas where the drainage systems are inadequate. Based on the magnitude, the rainfall is classified as Very light Rain (0.1- 2.4 mm/day), Light Rain (2.5 – 15.5 mm/day), Moderate Rain (15.6 – 64.4 mm/day), Heavy Rain (64.5 – 115.5 mm/day), Very Heavy Rain (115.6 – 204.4 mm/day) and Extremely Heavy Rain (>204.5 mm/day).

3.2.1 Measurement of Rainfall

Rainfall is measured by two types of gauges:

i. Self-recording type
ii. Non-recording type

Self-recording type rain gauges automatically record on a daily basis a continuous plot of rainfall depth against time down to 15 minutes interval or even less, whereas, non–recording rain gauges can only record cumulative rainfall for a day that is measured daily at site. The data collected using non-recording gauges are of limited use for design purpose. Hence, non-recording gauges are being gradually replaced in Indian subcontinent. Therefore, the mechanism of types of self – recording gauges that are in current use has been discussed as follows:

3.2.1.1 Tipping Bucket Type Rain Gauge

Tipping bucket type rain gauge is a 30 cm sized circular rain gauge adopted for use. It has 30 cm diameter sharp edged receiver and at the end of the receiver a funnel is provided.
Pair of buckets is pivoted under this funnel in such a manner that when one bucket receives 0.25 mm of precipitation (rainfall), it tips discharging its rainfall into the container, bringing the other bucket under the funnel as shown in Fig 3.1.

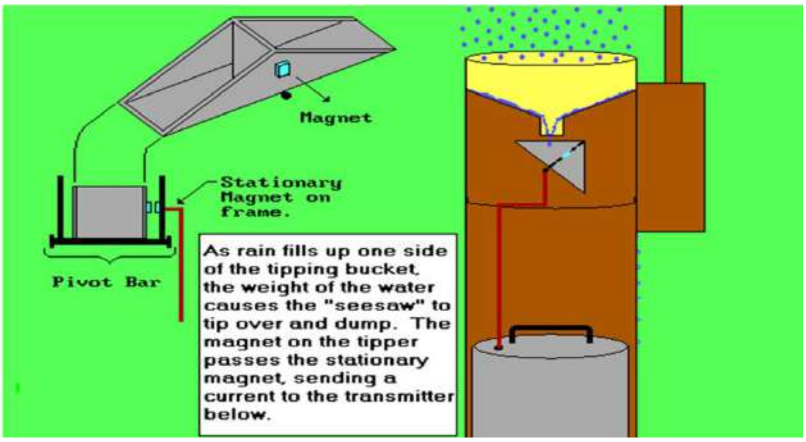


Figure 3.1: Tipping bucket type rain gauge

Tipping of bucket completes an electric circuit causing the movement of pen to mark on clock driven receiving drum which carries a recorded sheet. These electric pulses generated are recorded at the control room far away from the rain gauge station. This instrument is further suited for digitalizing the output signal.

3.2.1.2 Weighing Bucket Type Rain Gauge

Weighing bucket type rain gauge is most common self-recording rain gauge. It consists of a receiver bucket supported by a spring or lever balance or some other weighing mechanism. The movement of bucket due to its increasing weight is transmitted to a pen which traces record or some marking on a clock driven chart as shown in Fig 3.2.

Weighing bucket type rain gauge instrument gives a plot of the accumulated (increased) rainfall values against the elapsed time and the curve so formed is called the mass curve.

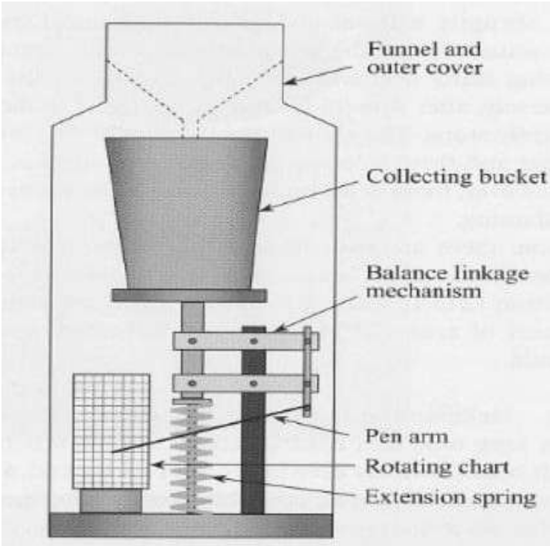


Figure 3.2: Weighing Bucket Type Rain Gauge

3.2.1.3 Floating or Natural Syphon Type Rain Gauge

The working of this type of rain gauge is similar to weighing bucket rain gauge. A funnel receives the water which is collected in a rectangular container. A float is provided at the bottom of container, and this float rises as the water level rises in the container. Its movement is recorded by a pen moving on a recording drum actuated by a clock work.

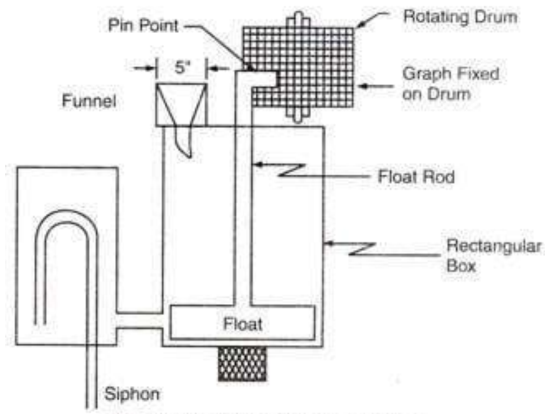


Figure 3.3: Natural Syphon or Float Type Rain Gauge

When water rises, this float reaches to the top, and then syphon comes into operation and releases the water outwards through the connecting pipe, thus all water in box is drained out. This rain gauge is adopted as the standard recording rain gauge in India and the curve drawn using this data is known as mass curve of rain fall.

Note: In most of the cases, IMD has installed Self – recoding Rain Gauge (SRRG) in various Cities and towns and the necessary desired/required data may be collected by the Project Implementing Agency. In case, SRRGs are installed by the States/Cities or any Agencies on their own, the necessary desired/required data may be facilitated to the Project Implementing Agencies, as per State Rules.

3.3 Rain Gauge Density

The rain gauge density in a catchment is defined as the ratio of total area of the catchment to the total number of rain gauge stations in the catchment. The term gives the average area served by each gauge. World Meteorological Organization, WMO (2008) has given guidelines regarding the minimum network density for urban areas as one rain gauge per 10 - 20 Sq.km. As per disaster management point of view, NDMA (National Disaster Management Authority) has recommended ARGs (Automatic Rain Gauge Stations) should be installed in all urban cities (Class I, II and III) with a density of 1 per 4 sq km.

Accordingly, following rain gauge density is recommended in urban areas:

- Population more than 10 Lakh : 1 rain gauge per 5 - 10 Sq.km.
- Population between 1 Lakh to 10 Lakh : 1 rain gauge per 10 - 20 Sq.km.
- Population less than 1 Lakh : 1 rain gauge per town.

3.4 Rainfall Analysis

Rainfall analysis is carried out to identify and sort out various magnitudes (intensities) of rainfall events and their corresponding durations occurring at a station from a continuous series of historic rainfall records taken for a fairly long period viz. last 25 - 30 years or more. IDF curves are not static as they are influenced by change in pattern of rainfall and therefore IDF curve should be prepared at an interval of 5 – 10 years for accurate results. The rainfall analysis helps to establish intensity-duration-frequency relationship for various frequencies which are used in estimation of runoff for design of storm water drains. The frequency or return period of a storm event may be defined as the average recurrence interval between events equal to or exceeding a specified magnitude. For example, if it is stated that a return period of a rainfall of magnitude 70 mm/hour at a station is 10 years that implies that on an average rainfall magnitude equals or exceeds 70 mm/hr once in 10 years.

3.4.1 Steps for Analysis of Rainfall

To illustrate the procedure of rainfall analysis, a continuous series of rainfall intensity and corresponding durations of historical storms of 29 years of Bhubaneshwar town is obtained from SRRG charts of each day from IMD rain gauge station at Bhubaneshwar. The data has been analysed for various return periods. Procedure is explained by the following steps:

STEP 1: The SRRG tabulated data may be obtained from IMD. In case, the tabulated data is not readily available then the SRRG Charts may be analysed to tabulate the data as explained with the help of a one day chart in the following Figure 3.4.

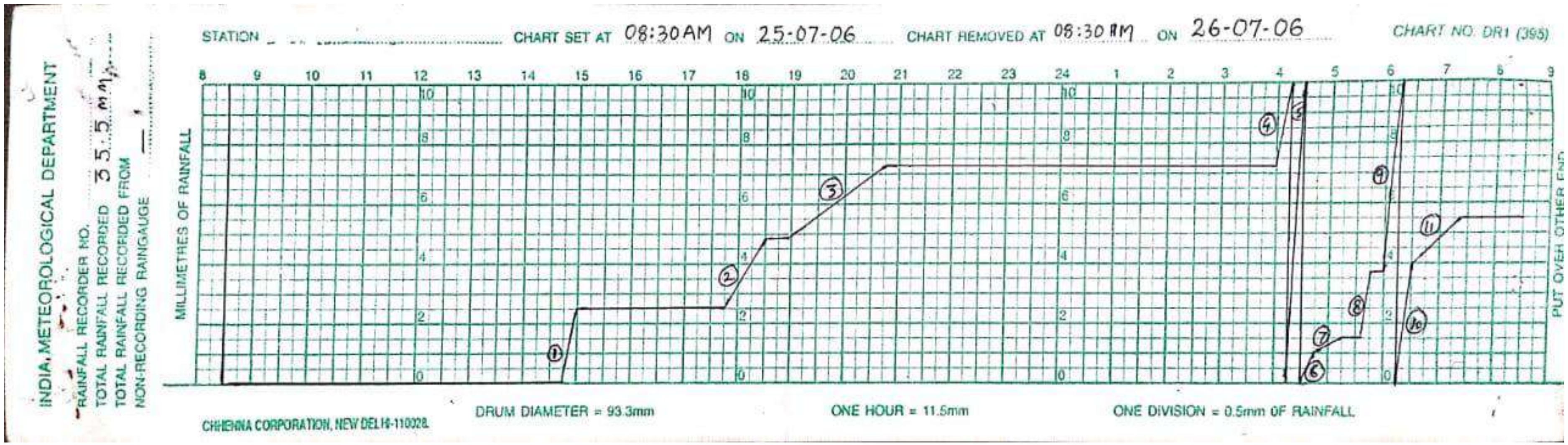


Figure 3.4: SRRG Chart for 24 hrs

From the sample SRRG chart given in Fig 3.4, tabulate rainfall of individual storm, its duration and find out intensity as given in the Table 3.1:

Table 3.1: Storm of intensities corresponding to duration

Year	Month	Date	Sr. No of Storms in Particular Day	No of Horizontal Divisions	Time in (minutes)	Time (Hours)	No of Vertical, Divisions	Rainfall in mm	Rainfall Intensity 'I' mm/hr
(a)	(b)	(c)	(d)	(e)	(f)	(g)= (f)/60	(h)	(i)=(h)x0.5	(j)= (i)/(g)
2006	July	25	1	1	15	0.25	5	2.50	10
			2	3	45	0.75	4.75	2.375	3.17
			3	7	105	1.75	4.75	2.375	1.36
			4	1	15	0.25	5.5	2.75	11.0
			5	1	15	0.25	20	10.0	40.0
			6	1	15	0.25	2	1.0	4.0
			7	2	30	0.5	1	0.5	1.0
			8	0.5	7.5	0.125	4.5	2.25	18.0
			9	1	15	0.25	12.5	6.25	25.0
			10	1	15	0.25	8.25	4.125	16.5
			11	3.5	52.5	0.875	2.75	1.375	1.57

Sort out the storms in various group of intensities corresponding to the duration of occurrence of storms**.** The number of storms are calculated and grouped in intensities of upto 5 mm/hr, 5 – 10 mm/hr, 10-15 m/hr and so on corresponding to each group of duration of occurrence as shown in the Table 3.2.

Table 3.2: Sorted storms against intensity and duration

	Duration in Mins	**Intensity (mm/Hr.)**															
		5-10	**10-15**	**15-20**	**20-25**	**25-30**	**30-35**	**35-40**	**40-45**	**45-50**	**50-55**	**55-60**	**60-75**	**75-90**	**90-120**	**120-150**	**>50**
Upto 5	5																
5 to 10	10			1													
10 to 15	15	1	1	1	1			1									
15 to 20	20																
20 to 25	25																
25 to 30	30																
30 to 40	40																
40 to 50	50																
50 to 60	60																
60 to 75	75																
75 to 90	90																
90 to 105	105																
105 to 120	120																

Note: Rainfall intensity below 5 mm/hr has not been taken for analysis.

STEP 2: Similarly, sort no of occurrences of rainfall intensities against corresponding duration for entire sample size of rainfall data obtained using MS Excel as shown in the Table 3.3.

Table 3.3: Sorted storms against intensity and duration

Duration in min	Intensity (mm/Hr.)															
	5-10	**10-15**	**15-20**	**20-25**	**25-30**	**30-35**	**35-40**	**40-45**	**45-50**	**50-55**	**55-60**	**60-75**	**75-90**	**90-120**	**120-150**	**>150**
Upto 5	35	13	22	4	11	1	2	3	5	2	4	3	3	1	1	1
5 to 10	149	40	57	23	14	4	7	6	7	2	7	4	4	1	1	
10 to 15	151	69	45	23	21	17	18	4	5	2	1	5	3	1	2	
15 to 20	4	2	6	0	1	0	1	0	0	0	0	1	0			
20 to 25	53	30	16	8	8	3	9	2	3	0	0	2	0			
25 to 30	89	53	37	32	22	5	11	5	5	3	0	0	0			
30 to 40	41	13	10	9	1	1	5	2	0	0	1	0	0			
40 to 50	81	41	23	9	11	6	9	2	2	3	0	0	1			
50 to 60	55	37	29	20	5	5	9	1	3	1	1	1	1			
60 to 75	32	21	16	10	14	5	3	2	1		1	1				
75 to 90	46	19	11	11	3	3	1	2			1					
90 to 105	30	13	4	4	5	2	0		2							
105 to 120	19	15	8	5	0	3	1									

STEP 3: Add number of storms of all exceeding intensities to the preceding /lesser intensities storms. For instance, in Table 3.3, number of storms corresponding to 5 min duration and various intensity groups i.e. 5 - 10 are added as 35 + 13 + 22 + 4+ 11 + 1 + 2 + 3 + 5 + 2 + 4 + 3 + 3 + 1 + 1 + 1 = 111. Similarly, number of storms are added horizontally for each duration of storms and tabulated in Table 3.4.

Table 3.4: Number of storms after horizontal addition

Duration in minutes	Number of Storms of Intensity (mm/Hr) or more															
	≥5	≥10	≥15	≥20	≥25	≥30	≥35	≥40	≥45	≥50	≥55	≥60	≥75	≥90	≥120	≥150
upto 5	111	76	63	41	37	26	25	23	20	15	13	9	6	3	2	1
5 to10	326	177	137	80	57	43	39	32	26	19	17	10	6	2	1	0
10 to 15	367	216	147	102	79	58	41	23	19	14	12	11	6	3	2	0
15 to 20	12	8	6	3	3	2	2	1	1	1	1	1	0	0	0	0
20 to 25	134	81	51	35	27	19	16	7	5	2	2	2	0	0	0	0
25 to 30	262	173	120	83	51	29	24	13	8	3	0	0	0	0	0	0
30 to 40	83	42	29	19	10	9	8	3	1	1	1	0	0	0	0	0
40 to 50	188	107	66	43	34	23	17	8	6	4	1	1	1	0	0	0
50 to 60	168	113	76	47	27	22	17	8	7	4	3	2	1	0	0	0
60 to 75	106	74	53	37	27	13	8	5	3	2	2	1	0	0	0	0
75 to90	97	51	32	21	10	7	4	3	1	1	1	0	0	0	0	0
90 to 105	60	30	17	13	9	4	2	2	2	0	0	0	0	0	0	0
105 to 120	51	32	17	9	4	4	1	0	0	0	0	0	0	0	0	0

STEP 4: Add number of storms of all exceeding durations to the preceding / lesser duration storms. For instance in Table 3.4, number of storms corresponding to ≥ 5 mm/hr intensity are added as 111+ 326+ 367+ 12+ 134+ 262+ 83+ 188+ 168+ 106+ 97+ 60+ 51=1965. Similarly, number of storms are added vertically for each Intensity and tabulated in Table 3.5.

Table 3.5: Number of storms after vertical addition

Duration in minutes	Number of Storms of Intensity (mm/Hr) or more for a period of 29 years															
	≥5	≥10	≥15	≥20	≥25	≥30	≥35	≥40	≥45	≥50	≥55	≥60	≥75	≥90	≥120	≥150
5	1965	1180	814	533	375	259	204	128	99	66	53	37	20	8	5	1
10	1854	1104	751	492	338	233	179	105	79	51	40	28	14	5	3	0
15	1528	927	614	412	281	190	140	73	53	32	23	18	8	3	2	0
20	1161	711	467	310	202	132	99	50	34	18	11	7	2	0	0	0
25	1149	703	461	307	199	130	97	49	33	17	10	6	2	0	0	0
30	1015	622	410	272	172	111	81	42	28	15	8	4	2	0	0	0
40	753	449	290	189	121	82	57	29	20	12	8	4	2	0	0	0
50	670	407	261	170	111	73	49	26	19	11	7	4	2	0	0	0
60	482	300	195	127	77	50	32	18	13	7	6	3	1	0	0	0
75	314	187	119	80	50	28	15	10	6	3	3	1	0	0	0	0
90	208	113	66	43	23	15	7	5	3	1	1	0	0	0	0	0
105	111	62	34	22	13	8	3	2	2	0	0	0	0	0	0	0
120	51	32	17	9	4	4	1	0	0	0	0	0	0	0	0	0

STEP 5: As given in Chapter 4, choose Design Return Period for the project area. Accordingly, determine number of storms allowed to exceed the design rainfall intensity (mm/hr). For example, the required numbers of storm events having intensity equal to or more than design intensity for once in 5 year occurrence for 29 years rainfall data will be 29/5 i.e. 5.8 times on an average may exceed over a period of 29 years. The same is given below for different design return periods:

a) Twice in a year = 29*2 = 58
b) Once in a year = 29*1 = 29
c) Once in 2 years = 29/2 = 14.5
d) Once in 5 years = 29/5 = 5.8

STEP 6: Draw a stepped line say for once in 5 year recurrence (5.8 no.) of occurrences occurring in intensity column and interpolate the corresponding duration (min) as shown in Table 3.6. Similarly, draw stepped lines for other return periods if required.

Table 3.6: Stepped line for number of storms for various storm return period

Duration in minutes	Number of Storms of Intensity (mm/Hr) or more for a period of 29 years															
	≥5	≥10	≥15	≥20	≥25	≥30	≥35	≥40	≥45	≥50	≥55	≥60	≥75	≥90	≥120	≥150
5	1965	1180	814	533	375	259	204	128	99	66	53	37	20	8	5	1
10	1854	1104	751	492	338	233	179	105	79	51	40	28	14	5	3	0
15	1528	927	614	412	281	190	140	73	53	32	23	18	8	3	2	0
20	1161	711	467	310	202	132	99	50	34	18	11	7	2	0	0	0
25	1149	703	461	307	199	130	97	49	33	17	10	6	2	0	0	0
30	1015	622	410	272	172	111	81	42	28	15	8	4	2	0	0	0
40	753	449	290	189	121	82	57	29	20	12	8	4	2	0	0	0
50	670	407	261	170	111	73	49	26	19	11	7	4	2	0	0	0
60	482	300	195	127	77	50	32	18	13	7	6	3	1	0	0	0
75	314	187	119	80	50	28	15	10	6	3	3	1	0	0	0	0
90	208	113	66	43	23	15	7	5	3	1	1	0	0	0	0	0
105	111	62	34	22	13	8	3	2	2	0	0	0	0	0	0	0
120	51	32	17	9	4	4	1	0	0	0	0	0	0	0	0	0

STEP 7: Intensity duration as interpolated in Step 6 is given in the Table 3.7.

Table 3.7: Intensity Duration for Storm of once in 5 year

Duration (min)	Intensity (mm/hr)
116.83	25
112.87	30
94.12	35
87.3	40
75.5	45
64.12	50
60.5	55
25.25	60
16.75	75
8.5	90

STEP 8: Establish Intensity Duration Frequency relationship

IDF relationship formulae are empirical ones that were developed based on observation that as the time duration of storm increases the intensity of storm decreases. Bernard equation is normally adopted i.e. $i = \frac{a}{t^n}$ for Indian conditions. The constants of the equation are found out by curve fitting technique which is described as follows:

The equation i = $\frac{a}{t^n}$ on logarithmic scale turns into the following form which is a straight line equation,

$$\log\ i = \log a - n \log t \qquad (3.1)$$

Where,

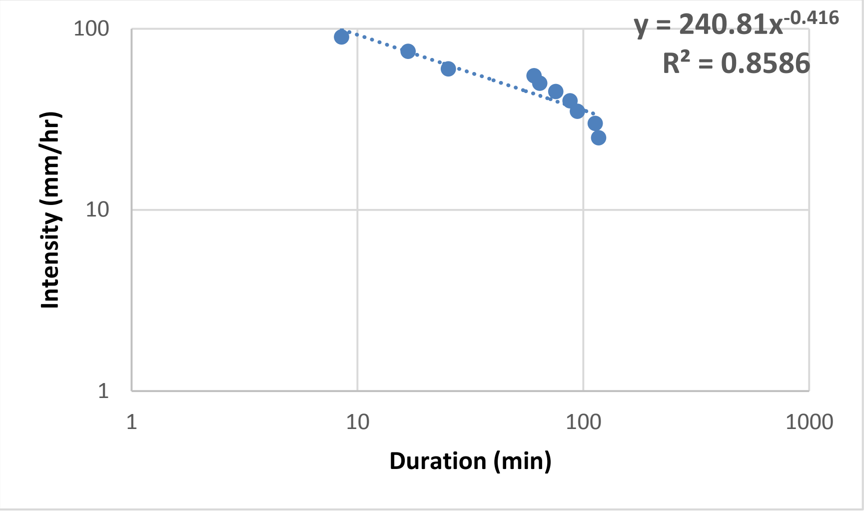
i is intensity of rainfall (mm/hr)
t is rainfall duration (min)
a and n are constant

Thus by plotting i and t on log- log graph paper, the trend line can be approximated to a straight line of best fit. The slope of this line will give the value of 'n' and its intercept on Y axis will give the value of 'a'.

For example, Intensities durations analysed for 5 year return period for Bhubaneswar town as tabulated and given in the Table 3.7 is plotted on log – log paper. Constants 'a' and 'n' are determined.

Table 3.8: Log – log graph between Intensity Duration for Storm Return Period for once in 5 year

Duration (min)	Intensity (I) (mm/hr)
116.83	25
112.87	30
94.12	35
87.3	40
75.5	45
64.12	50
60.5	55
25.25	60
16.75	75
8.5	90

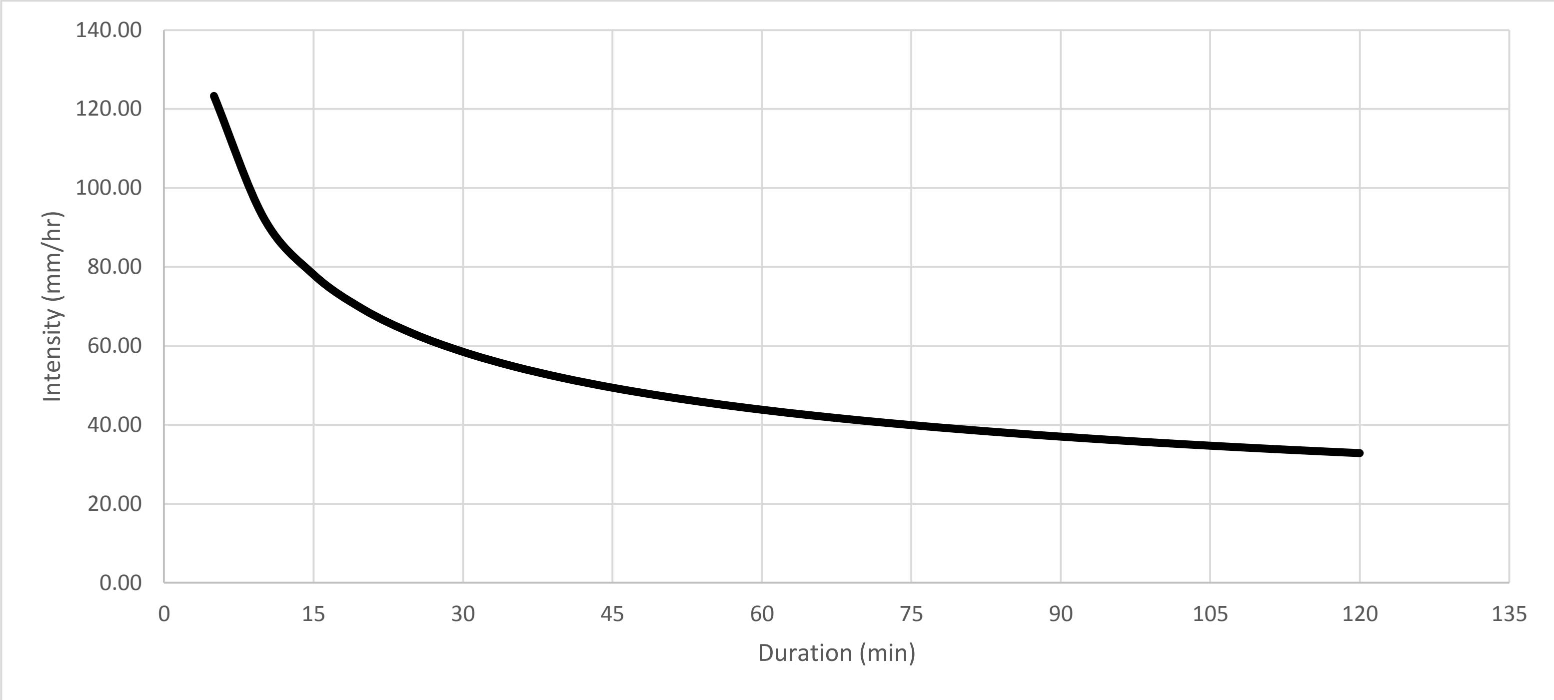


From the log – log graph, **a and n** values can be read as **240.81 and 0.416** respectively.

STEP 9: After the constants are determined, the intensities for various durations such as 5, 10, 15, 20, minutes and so on can be determined by Bernald Equation i.e. $i = \frac{a}{t^n}$. Intensities and durations so computed are plotted on arithmetic graph paper and joined with smooth curve to trace the IDF curve of given frequency as carried out in Table 3.9.

Table 3.9: IDF for Storm of Once in a 5 Year

Duration (min)	Intensity (mm/hr)
5	123.28
10	92.40
15	78.06
20	69.25
25	63.11
30	58.50
35	54.87
40	51.91
45	49.42
50	47.30
60	43.85
70	41.13
80	38.90
90	37.04
100	35.45
110	34.08
120	32.87



Similarly, IDF curves for other return periods are prepared.

3.5 Alternative Method of Rainfall Analysis

In case rainfall data of depth – duration are available in successive 15 min intervals from IMD, then the method of analysis can also be adopted using following steps:

1. Collect continuous observed rainfall data in successive 15 min intervals from IMD for a fairly long period (minimum 25-30 years) or more.
2. Analyse one rainfall event into depth and duration for 15 min, 30 min, 45 min and so on as analysed in the following example given in Table 3.10 for 90 min rainfall event.

Table 3.10: Rainfall Analysis of single storm

Duration (min)	**Rainfall Depth (mm)**					
15	**4**	**8**	**16.5**	**11.5**	**7**	**10.5**
30	**12** (4+8 =12)	**24.5** (8+16.5 = 24.5)	**28** (16.5+11.5 = 28)	**18.5** (11.5+7 = 18.5)	**17.5** (7+10.5=17.5)	
45	**28.5** (4+8+16.5=28.5)	**36** (8+16.5+11.5 =36)	**35** (16.5+11.5+7=35)	**29** (11.5+7+10.5)		
60	**40** (4+8+16.5+11.5= 40)	**43** (8+16.5+11.5+7 = 43)	**45.5** (16.5+11.5+7+10.5= 45.5)			
75	**47** (4+8+16.5+11.5+7=47)	**53.5** (8+16.5+11.5+7+10.5 = 53.5)				
90	**57.5** (4+8+16.5+11.5+7+10.5=57.5)					

Similarly, analyse all rainfall events into depths and corresponding duration occurring during the entire sample size and convert the depths into intensity.

3. Sort out and tabulate number of storms of various intensities as analysed above for corresponding durations and from the observed storm events of entire sample size as shown in table 3.11.

Table 3.11: Number of storms of intensities against corresponding duration

Duration (min)	**10<Int< 20 mm/hr**	**20<Int <=30 mm/hr**	**30<Int <=40 mm/hr**	**40<Int <=50 mm/hr**	**50<Int <=60 mm/hr**	**60<Int <=70 mm/hr**	**70<Int <=80 mm/hr**	**80<Int <=90 mm/hr**	**90<Int <=100 mm/hr**	**100<Int <=110 mm/hr**	**110<Int <=120 mm/hr**	**120<Int <=130 mm/hr**	**Int > 130 mm/hr**
	No. of storms of intensity for 25 Year												
15	419	263	165	76	44	24	33	8	2	2	2	1	1
30	126	130	87	49	30	16	10	3	3	1	2		
45	48	54	54	24	14	11	1	2	3				
60	31	26	25	19	7	4	1	2					
75	18	6	15	11	5	1	1						
90	8	3	8	9	4	1							

4. Add the number of occurrences of Rainfall Intensities equal or exceeded against corresponding duration. For instance in Table 3.11, number of storms corresponding to 15 min duration is added as 419 + 263+ 165+ 76+ 44+ 24+ 33+ 8+2+ 2+ 2+ 1+ 1=1040.

Table 3.12: Number of storms of intensity or more against corresponding duration

Duration (min)	**10**	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**	**100**	**110**	**120**	**130**
	No. of storms of intensity or more for a period 25 Year												
15	1040	621	358	193	117	73	49	16	8	6	4	2	1
30	457	331	201	114	65	35	19	9	6	3	2		
45	211	163	109	55	31	17	6	5	3				
60	115	84	58	33	14	7	3	2					
75	57	39	33	18	7	2	1						
90	33	25	22	14	5	1							

5. Subsequently, procedure for IDF curve preparation is same as given in Step 5 to Step 9 of section 3.4.1

3.6 Probabilistic Method

Probabilistic method based on probability distribution for random hydrologic variables is an approach for predicting probability of flood flows, rainfall etc. of observed large sample size that are statistically analysed to predict the extreme future events of desired frequency. Various Methods are as follows:

a) Normal Distribution
b) Log Normal Distribution
c) Gumbel Extreme Value Distribution
d) Log Pearson Type III Distribution

3.6.1 Normal Distribution and Log Normal Distribution

The normal and log normal distribution can only give good results if the skewness coefficient of data series is equal to zero. As rainfall data can hardly comply with these conditions, hence it is commonly not applied for frequency analysis of such data.

3.6.2 Gumbel Extreme Value Distribution

The extreme value distribution introduced by Gumbel is commonly known as Gumbel distribution. It is widely used probability distribution function for extreme values in hydrologic and meteorological studies for prediction of flood peak and maximum rainfall etc. Gumbel distribution is widely used in Indian sub-continent. Therefore, the process of the analysis is described below:

The equation is given as

$$X_T = u + \alpha\, y_T \tag{3.2}$$

Where, u and α are the mode of distribution and sample moments respectively which is given by the following equation.

$$u = \overline{X} - 0.5772\alpha \tag{3.3}$$

$$\alpha = (\sqrt{6} / \pi) S_x \tag{3.4}$$

A reduced variate y_T for a return period can be defined as

$$y_T = -ln\left[ln\left(\frac{T}{T-1}\right)\right] \tag{3.5}$$

Where,

X_T = T- year return period value
$\bar{X}$ = Mean of the N observations
S_x = Standard deviation of N observations $= \sqrt{\frac{(x-\bar{x})^2}{N-1}}$
X = Rainfall Event
T = Recurrence interval (Storm Return Period)
N = Sample size

3.6.2.1 Construction of IDF curve by Gumbel Distribution method

Rainfall Data has been obtained from IMD of the Safdarjung rain gauge station. Following steps are to be taken for the construction of IDF Curve by Gumbel Method:

Step 1: Determine the maximum depth of rainfall of each rainfall event for 15,30,45,60.....minutes interval occurring on one day i.e. 25.7.1982 as given in the Table 3.13

Table 3.13: Maximum rainfall depth

Duration (min)	Rainfall Depth (mm)						Max Rainfall (mm)
15	4	8	16.5	11.5	7	10.5	16.5
30	12	24.5	28	18.5	17.5		28
45	28.5	36	35	29			36
60	40	43	45.5				45.5
75	47	53.5					53.5
90	57.5						57.5

Similarly, determine maximum rainfall depth and duration for all rainfall events occurring each day for the entire year and then find out the maximum rainfall depth and duration occurring in the year for 15,30,45…minutes for 25 years.

Step-2: Similarly, maximum rain fall depths of each year for 15,30,45,60.......minutes duration is obtained for entire sample size i.e 25 years and thus annual maximum series has been prepared as given in the Table 3.14.

Table 3.14: Maximum annual series Rainfall Depth (mm)

Year	15 min	30 min	45 min	60 min	75 min	90 min
1979	21.5	25.5	32.5			
1980	16.5	26	38	40.2	25.3	28.6
1981	13.5	18	24	19.5	23	26
1982	21	37.5	43.2	47	53.5	57.5
1983	10.6	18.6	16.6	20.3		
1984	27	41.5	58			
1985	18.8	34.8	34	45	49	
1986	21.5	41	16.3	19.5	23.3	
1987	18.5	26				
1988	20.2	22.7	20.5	23.5		
1989	22	40	47.5	52.8	42	45
1990	35.8	55.8	85.8	109.8	125.8	135.8
1991	20.7	27.5	34	44	50.5	
1992	22	34	38.2	40.2	39	
1993	18.5	26	30	36.5	41.5	50
1994	41	56	61.5	56		
1995	19	30.5	40.5	45		
1996	18	36	50	61	17.3	19.8
1997	34	25	34	38.3	21	23
1998	30	50	70	82	86.5	91.5
1999	18					
2000	20	32.5	50.3	60.3	55	61.5
2001	27	28	47	53		
2002	29	30	32.5	15.2	19	23.6
2003	30	40	37.8	28		

Step-3: Gumbel distribution is applied on the above tabulated annual series to obtain maximum values for annual rainfall depth corresponding to 15,30,45,60....minutes duration for 5 years storm return period and subsequently converted into intensity as shown in the table 3.15.

Table 3.15: Computation using Gumbel distribution method

	15 min	30 min	45 min	60 min	75 min	90 min
Mean ($\bar{X}$)	22.96	33.45	40.97	44.62	44.78	51.12
Standard Deviation (S_x)	7.12	10.38	16.74	22.38	29.14	35.57
$\alpha = (\sqrt{6}/\pi)S_x$	5.549	8.09	13.05	17.44	22.71	27.72
$u = \overline{X} - 0.5772\alpha$	19.76	28.78	33.44	34.55	31.67	35.12
For T = 5 years						
$y_T = -ln\left[ln\left(\frac{T}{T-1}\right)\right]$	1.5	1.5	1.5	1.5	1.5	1.5
$X_T = u + \alpha\, y_T$	28.08	40.92	53.01	60.72	65.74	76.7
Intensity in mm/hr	112.3	81.83	70.68	60.72	52.59	51.13

Step 4: Plot Intensity Duration Frequency for the above obtained values:

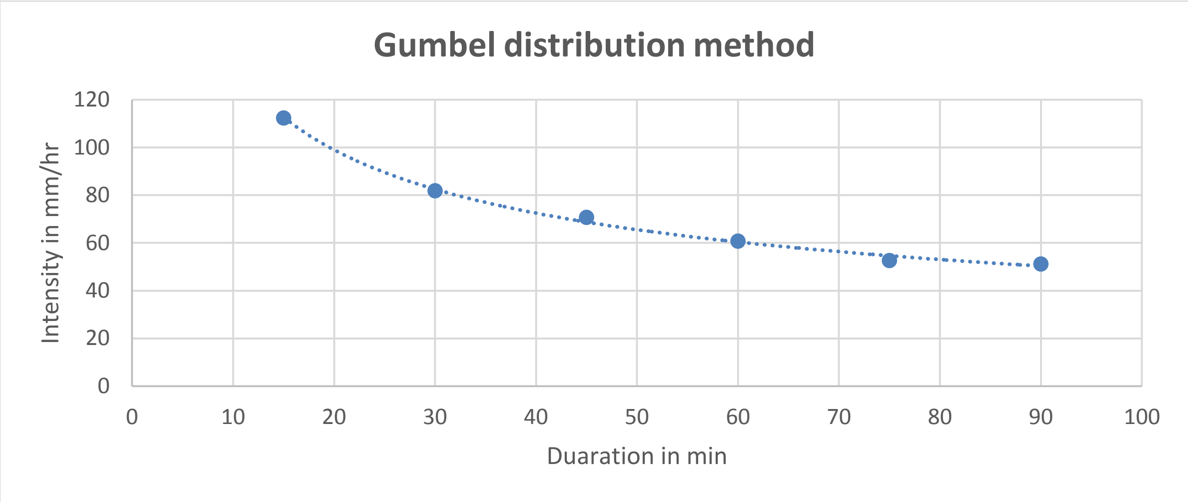


Figure 3.5: IDF curve for 5 year Return Period

3.6.3 Log Pearson Type III Distribution

Log Pearson type III distribution is widely used for frequency analysis for stream flows and can also be used for rainfall. The values obtained by Log Pearson type III distribution is more satisfactory as it has 3 parameter distribution that considers mean, standard deviation and skewness of data series. Process of computations described as follows:

- The variate (data series) is transformed into logarithmic form either on base (10 or e) and the transformed data is then analysed.
- If X is the variate of random hydrologic series then the series of z variates where z = log X

For this z series, for any recurrence interval, T, the equation

$$z_T = \bar{z} + K_z \sigma \qquad (3.6)$$

Where,

K_z : Frequency factor which is function of recurrence interval T and coefficient of skew C_s

σ : Standard deviation of z variate sample $\left[\frac{\sum(z-\bar{z})^2}{N-1}\right]^{1/2}$

C_s: Coefficient of skew of variate z

$= N\left[\frac{\sum(z-\bar{z})^3}{(N-1)(N-2)\sigma^3}\right]^{1/3}$

$\bar{z}$: Mean of the z values

N = sample size

The variation of K_z = f (C_s , T) is given in the Table 3.16 and Table 3.17

After finding out z_T, corresponding value of X_T can be obtained by taking antilog of z_T

Table 3.16: K_z values for Pearson Type III distribution (Positive Skew)

Skew coefficient C_s	Return period in years						
	2	5	10	25	50	100	200
	Exceedance probability						
	0.50	0.20	0.10	0.04	0.02	0.01	0.005
3.0	-0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.9	-0.390	0.440	1.195	2.277	3.134	4.013	4.909
2.8	-0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.7	-0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.6	-0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.5	-0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.4	-0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.3	-0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.2	-0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.1	-0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.0	-0.307	0.609	1.302	2.219	2.912	3.605	4.298
1.9	-0.294	0.627	1.310	2.207	2.881	3.553	4.223
1.8	-0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.7	-0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.6	-0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.5	-0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.4	-0.225	0.705	1.337	2.128	2.706	3.271	3.828

1.3	-0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.2	-0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.1	-0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.0	-0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.9	-0.148	0.769	1.339	2.018	2.498	2.957	3.401
0.8	-0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.7	-0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.6	-0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.5	-0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.4	-0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.3	-0.050	0.824	1.309	1.849	2.211	2.544	2.856
0.2	-0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.1	-0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.0	0	0.842	1.282	1.751	2.054	2.326	2.576

Table 3.17: K_z values for Pearson Type III distribution (Negative Skew)

Skew coefficient C_s	Return period in years						
	2	**5**	**10**	**25**	**50**	**100**	**200**
	Exceedance probability						
	0.50	**0.20**	**0.10**	**0.04**	**0.02**	**0.01**	**0.005**
-0.1	0.017	0.846	1.270	1.716	2.000	2.252	2.482
-0.2	0.033	0.850	1.258	1.680	1.945	2.178	2.388
-0.3	0.050	0.853	1.245	1.643	1.890	2.104	2.294
-0.4	0.066	0.855	1.231	1.606	1.834	2.029	2.201
-0.5	0.083	0.856	1.216	1.567	1.777	1.955	2.108
-0.6	0.099	0.857	1.200	1.528	1.720	1.880	2.016
-0.7	0.116	0.857	1.183	1.488	1.663	1.806	1.926
-0.8	0.132	0.856	1.166	1.448	1.606	1.733	1.837
-0.9	0.148	0.854	1.147	1.407	1.549	1.660	1.749
-1.0	0.164	0.852	1.128	1.366	1.492	1.588	1.664
-1.1	0.180	0.848	1.107	1.324	1.435	1.518	1.581
-1.2	0.195	0.844	1.086	1.282	1.379	1.449	1.501
-1.3	0.210	0.838	1.064	1.240	1.324	1.383	1.424
-1.4	0.225	0.832	1.041	1.198	1.270	1.318	1.351
-1.5	0.240	0.825	1.018	1.157	1.217	1.256	1.282
-1.6	0.254	0.817	0.994	1.116	1.166	1.197	1.216
-1.7	0.268	0.808	0.970	1.075	1.116	1.140	1.155
-1.8	0.282	0.799	0.945	1.035	1.069	1.087	1.097
-1.9	0.294	0.788	0.920	0.996	1.023	1.037	1.044
-2.0	0.307	0.777	0.895	0.959	0.980	0.990	0.995
-2.1	0.319	0.765	0.869	0.923	0.939	0.946	0.949
-2.2	0.330	0.752	0.844	0.888	0.900	0.905	0.907
-2.3	0.341	0.739	0.819	0.855	0.864	0.867	0.869
-2.4	0.351	0.725	0.795	0.823	0.830	0.832	0.833

-2.5	0.360	0.711	0.771	0.793	0.798	0.799	0.800
-2.6	0.368	0.696	0.747	0.764	0.768	0.769	0.769
-2.7	0.376	0.681	0.724	0.738	0.740	0.740	0.741
-2.8	0.384	0.666	0.702	0.712	0.714	0.714	0.714
-2.9	0.390	0.651	0.681	0.683	0.689	0.690	0.690
-3.0	0.396	0.636	0.666	0.666	0.666	0.667	0.667

3.6.3.1 Construction of IDF curve by Log Pearson type III method

The same data series which has been analysed for Gumbel distribution as given in Table 3.14 has been used for Log Pearson type III method.

The data series has been transformed in logarithmic series and the computation is done as follows:

Table 3.18: Computation using Log Pearson type III method

	15 min	**30 min**	**45 min**	**60 min**	**75 min**	**90 min**
Mean ($\bar{z}$)	3.0894	3.4656	3.6323	3.6818	3.6385	3.7465
Standard Deviation (σ)	0.305	0.304	0.418	0.503	0.575	0.624
Coefficient of Skewness (C_w)	0.011	-0.031	0.722	1.054	-2.791	-2.593
T = 5 years						
K from WRC 1981 with Coefficient of Skewness (C_s)	0.84134	0.84014	0.7878	0.75098	0.66735	0.69735
$z_T = \bar{z} + K_z\sigma$	3.34601	3.721	3.9616	4.05954	4.02223	4.18165
X_T = exp ($\bar{z} + K_z\sigma$)	28.389	41.306	52.541	57.948	55.825	65.474
Intensity in mm/hr	113.56	82.61	70.06	57.95	44.66	43.65

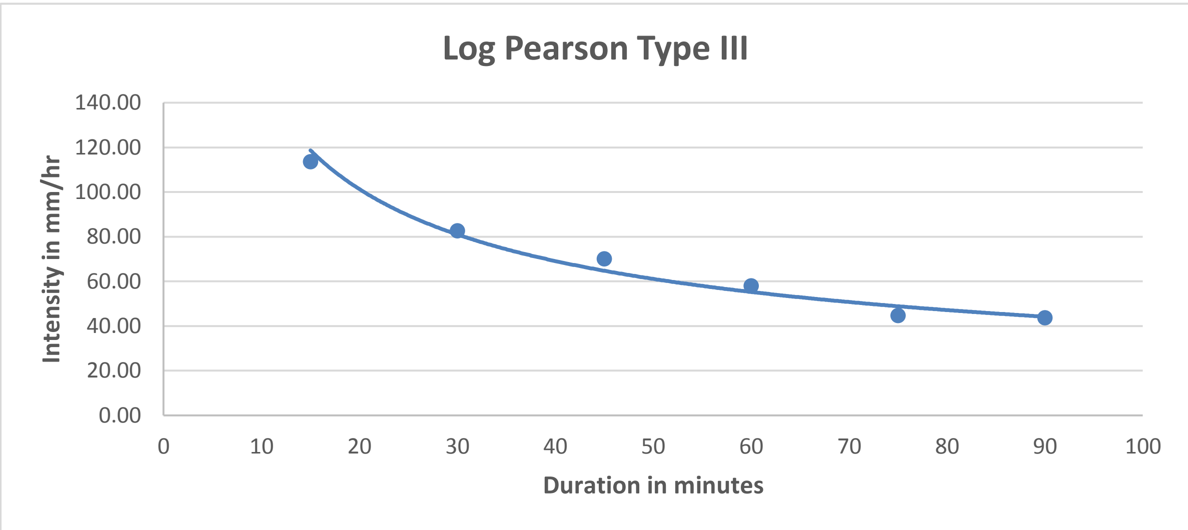


Fig 3.6: IDF curve from Log Pearson Type III

3.7 Translation of IDF curve into rainfall hyetograph

Hyetograph is a plot of rainfall depth against the time duration. It is usually represented as bar chart. The Hyetograph translated from IDF curve can be used in SWMM model for calculating runoff from the catchment for which IDF curve is constructed. This can also be superimposed upon unit Hydrograph for a given catchment to compute the direct runoff hydrograph provided the IDF curve relates to that catchment. Time area method also uses Hyetograph for computation of runoff from the catchment.
The alternating block method is a simple way to develop a rainfall hyetograph from an intensity duration curve for a given storm return period. The rainfall hyetograph generated by this method specifies the precipitation depth occurring in n successive time intervals of duration δ_t over a total duration of $T_d = n\delta$. The intensity from a given return period is read from the IDF curve for each of the duration and corresponding precipitation depth is found as product of intensity and duration. By taking difference between successive precipitation depth values, the amount of precipitation to be added for each additional unit of time δ_t is found. These increments or blocks are recorded into a time sequence with maximum intensity occurring at the centre of the required duration T_d and the remaining blocks are arranged in descending order alternately to the right and left of the central block to form the required rainfall hyetograph as shown in the Table 3.19.

Using the data of Intensity and duration, a sample Hyetograph is prepared as follows:

Table 3.19: Computation to prepare Hyetograph

Duration (minutes)	Intensity (mm/hr)	Successive Depth. (mm)	Incremental Depth (mm)		Time (Minutes)	Precipitation (mm)
10	151.38	25.23	25.23		0-10	4.27
20	108.61	36.20	10.97		10-20	4.71
30	89.44	44.72	8.52		20-30	5.36
40	77.92	51.95	7.23		30-40	6.41
50	70.03	58.36	6.41		40-50	8.52
60	64.17	64.17	5.81		50-60	25.23
70	59.6	69.53	5.36		60-70	10.97
80	55.91	74.55	5.01		70-80	7.23
90	52.84	79.26	4.71		80-90	5.81
100	50.24	83.73	4.47		90-100	5.01
110	48	88.00	4.27		100-110	4.47
120	46.04	92.08	4.08		110-120	4.08

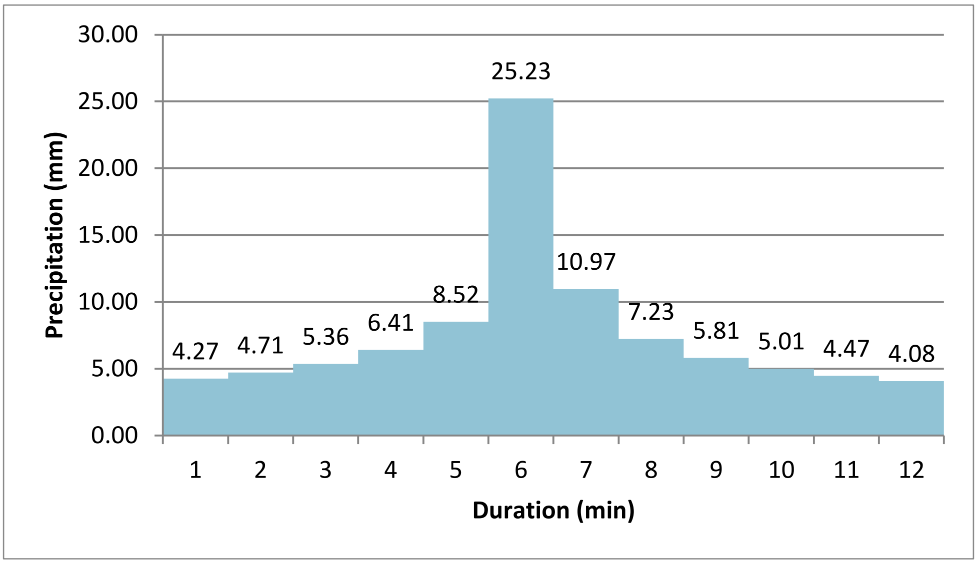


Figure 3.7: Hyetograph

CHAPTER 4: RUNOFF ESTIMATION

4.1 General

The basic requirement for designing of Storm Water Drainage system is the proper estimation of storm runoff to downstream drains or to the point of disposal. It has bearing on optimizing cost of infrastructure as well as its performance. The parameters like rainfall intensity, imperviousness factor, coefficient of runoff, recurrence period, climate change and identification / zoning of drainage catchment play an important role. In chapter 3, the analysis of rainfall has been dealt in detail. In this chapter various methods of estimation of storm runoff like Rational Method, Time Area Method, Unit Hydrograph Method and Rainfall-Runoff Simulation method are explained.

4.2 Storm Runoff

Runoff from a catchment is that fraction of precipitation which generates surface flow. It thus represents the output from the catchment corresponding to precipitation in a given unit of time. For a given precipitation, initial losses due to the interception, evapo-transpiration, infiltration and detention storage requirements have to be first satisfied before commencement of runoff. After these losses are met, the excess rainfall moves over the surface termed as storm runoff. This is illustrated in Figure 4.1 below.

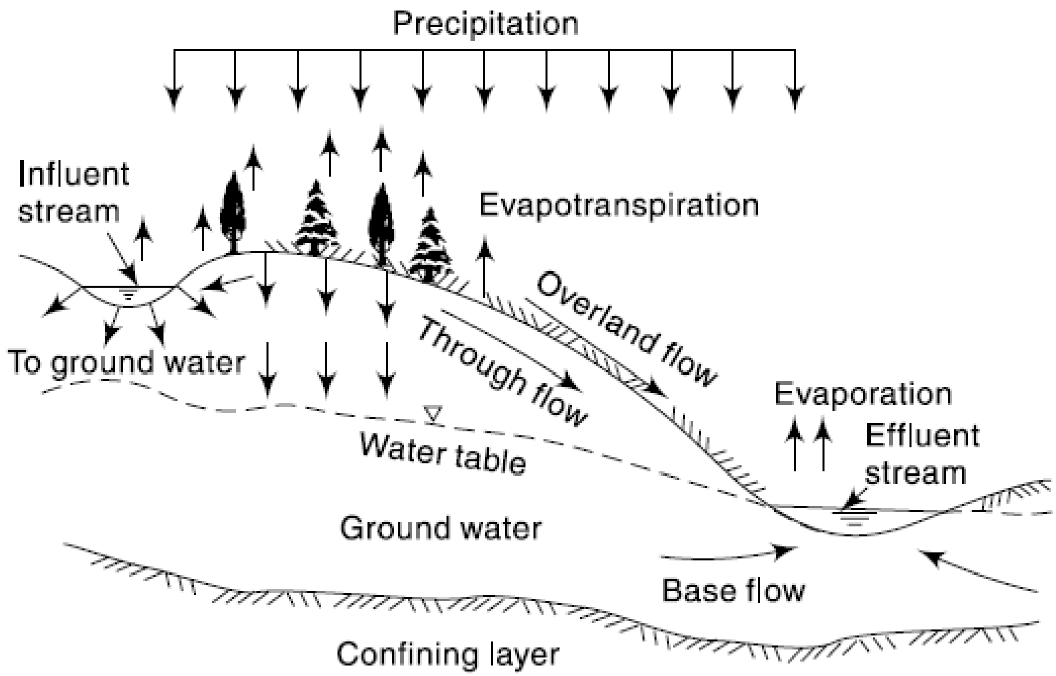


Figure 4.1:Different routes of runoff

4.3 Estimation of runoff from rainfall

The runoff estimation is affected by following factors of catchment hydrology:

- Size of Catchment
- Shape of the Catchment i.e. Fan shaped, Fern shaped, Irregular Shaped etc.
- Elevation of the Catchment
- Drainage Density
- Type of soil of the catchment
- Type of cover viz. paved, unpaved, vegetative etc.
- Slope and orientation of the catchment
- Topography (Depression storages / ponds / lakes) and geology of the catchment
- Saturation of soil with water due to previous precipitation if any, including the level of ground water table.

4.4 Methods of runoff estimation

The following methods are generally used for runoff estimation for design of urban storm water drainage systems.

1. Rational Method
2. Time Area Method
3. Unit Hydrograph Method
4. Rainfall- Runoff process simulation

The above methods and their use in design of storm water networks are given below.

4.4.1 Rational Method

The rational method was developed during the second half of the 19th century for estimating design discharge from an urban catchment. Majority of urban storm drainage systems are designed on the basis of Rational Method, in as much as 90% cases across the globe, in spite of having several limitations.

4.4.1.1 Steps of computation for Rational method

The procedure for estimation of storm runoff by rational method is mentioned in the following steps:

Step 1: Obtain historical rainfall data of 30 years or more for the given project area

Step 2: Select a return period from the Table 4.1 as required

Step 3: Prepare IDF curve for the above return period as per Chapter 3

Step 4: Discretize the catchment by demarcating watersheds and drainage patterns of the project area with the aid of existing maps and contour plans

Step 5: Determine time of concentration (t_c) as described in section 4.4.1.5

Step 6: Determine rainfall intensity against time of concentration from IDF curve

Step 7: Determine runoff coefficient (C) as described in section 4.4.1.4

Step 8: Calculate peak flow by Rational formula as given in section 4.4.1.2

4.4.1.2 Design flow

Storm water drains are designed taking into account the peak flow. The peak flow is defined as the flow when the entire catchment is contributing at its outlet. This will occur when the given intensity of rainfall begins instantaneously and continues till the time of concentration.

4.4.1.3 Procedure for estimation of runoff

If properly understood and applied, the 'rational method' can produce satisfactory results for sizing storm drains, street inlets and small on-site detention catchments. The formula for calculating design flow is given as below:

$$Q_p = 10\ C\ I\ A \qquad (4.1)$$

Where,

Q_p : Peak flow at the point of design, m^3/hr

C : Runoff coefficient, dimensionless

I : Average rainfall intensity should be taken for the duration of rainfall equal to the time of concentration, mm/hr

A : Catchment area, hectares

This formula is dimensionally consistent to other measurement systems.

Although this method is widely used in storm water drainage design, the estimation of runoff involves the following assumptions:

a) The maximum size of a catchment should be between 8 to 10 sq km.
b) Larger catchments can be sub-divided into smaller sub-catchments.
c) The peak flow occurs when the entire catchment is contributing to the flow;
d) The rainfall intensity is uniform over the entire catchment;
e) The rainfall intensity is uniform over a time duration equal to the time of concentration;
f) The frequency of the computed peak flow is the same as that of the rainfall intensity corresponding to the return period of the 'design storm';
g) The coefficient of runoff is the same for all storms of all recurrence probabilities.

4.4.1.4 Design Return Period

The design return period of a storm is an average period of time after which it reoccurs, for a given rainfall intensity or more and corresponding to a particular duration of time. This is integral part of IDF curve developed, based on analysis of past rainfall data, for designing of storm water drainage systems. Depending on importance of the drainage area, socio-economic conditions of the city and other constraints such as funding for infrastructure and availability of space for construction of drains, the design return period of storm should be judiciously adopted in estimation of storm runoff. In view of the above, the recommended design return period of storms is given in Table 4.1 for estimation of storm runoff.

Table 4.1: Recommended Design Return Period for various types of urban catchments

S. No.	Urban Catchment	Return Period	
		Mega Cities	Other cities
1.	Central Business and commercial	Once in 5 years	Once in 2 years
2.	Industrial	Once in 5 years	Once in 2 years
3.	Urban Residential Core Area, Peripheral Area	 Once in 5 years Once in 2 years	 Once in 2 years Once in 1 years
4.	Open space, Parks and landscape	Once in 6 months	Once in 6 months
5.	Airports and other critical infrastructure*	Once in 100 years	Once in 50 years

*critical infrastructure includes Railway Stations, Power stations, etc.

Note:

1. It may not be always feasible to design / retrofit the storm water drains for the recommended return period in all the cities. In cases where redesigning / retrofitting is not feasible as per recommended return period due to city profile / site constraints, efforts should be made to adopt recommended return period by adopting 'Best Management Practices, (BMP) like rainwater harvesting and storm retention/ detention structures to accommodate the

excess runoff. However, the preferred return period shall be as per those recommended in the Table 4.1 above.

2. The design performance of above drains can be further raised to higher return period say once in 10 years, in case of commercial and high importance area having high frequency of flooding by adopting rainwater harvesting structures suggested in the Manual at household levels / in-situ / along the storm water drains / conduits.

3. In case, it is very much necessary for a city or state to adopt higher return period for construction of storm water conveyance and disposal system due to various design considerations and site specific requirements, the same can be permitted after approval of the Principal Secretary, in-charge in the state, subject to condition that additional cost of project over and above the one based on recommended return period in Table 4.1 has to be borne by city / state government.

4.4.1.5 Runoff Coefficient

The coefficient of runoff (C), is a function of the nature of surface and assumed to be the same for all storms of all recurrence probabilities. Recommended values of C on various surface types of catchment are given in Table 4.3. While choosing the values for C, the ultimate development of the catchment as per the master plan should be taken into consideration.

The percentage of imperviousness of the drainage area can be obtained from the records of a particular district. In the absence of such data, Table 4.2 may serve as a guide.

Table 4.2: Percentage of Imperviousness of Areas

S. No.	Type of Area	Percentage of Imperviousness
1	Commercial and Industrial Area	70 - 90
2	Residential Area -High Density -Low Density	 61 – 75 35 - 60
3	Parks and undeveloped areas	10 - 20

Source: Manual on Sewerage & Sewage Treatment Plants, CPHEEO, 2013

When several different surface types or land use comprise the drainage area, a composite or weighted average value of the imperviousness runoff coefficient can be computed, such as:

$$I_m = [(A_1 I_{m1}) + (A_2 I_{m2}) + \ldots\ldots (A_n I_{mn})]/[A_1 + A_2 + \ldots\ldots + A_n] \quad (4.2)$$

Where,
I_m : Weighted average imperviousness of the total drainage catchment
A_1, A_2, A_n : Sub drainage areas
I_{m1},I_{m2},I_{mn} : Imperviousness of the respective sub-areas.

Example 4.1:

If there is a catchment of 30 hectare area and each 10 hectare areas is having imperviousness factor 0.6, 0.3 and 0.7. Find out the weighted average imperviousness of the entire catchment of 30 hectares.

Solution:
A_1 = 10 hectare ; Imperviousness (I_{m1}) = 0.6
A_2 = 10 hectare ; Imperviousness (I_{m2}) = 0.3
A_3 = 10 hectare ; Imperviousness (I_{m3}) = 0.7

Weighted average value of the imperviousness runoff coefficient can be computed, such as:
$I = ((A_1I_{m1} + A_2I_{m2} + A_3I_{m3})/(A_1+A_2+A_3)$
$I = (10*0.6+10*0.3+10*0.7)/(10+10+10)$
$I = 0.53$
In percent I = 53 %
Either, value of C should be interpolated or the catchment may be converted to 100 % imperviousness as follows:
Area of the catchment × composite imperviousness factor = 30 × 0.53 = 15.9 Hectare
For this area, 100 % imperviousness factor should be taken for finding out the Value of C from the Table 4.3.

The weighted average runoff coefficients for rectangular areas, of length four times the width as well as for sector shaped areas with varying percentages of impervious surface for different time of concentration are given in Table 4.3.

Although these are applicable to particular shape areas, they also apply in a general way to the areas, which are usually encountered in practice. Errors due to difference in shape of drainage are within the limits of accuracy of the rational method and of the assumptions on which it is based.

Table 4.3: Runoff coefficients for times of concentration

Duration, t, minutes	10	20	30	45	60	75	90	100	120	135	150	180
Weighted Average Coefficient												
1. Sector concentrating in stated time												
a. Impervious	0.525	0.588	0.642	0.70	0.740	0.771	0.795	0.813	0.828	0.840	0.850	0.865
b. 60% Impervious	0.365	0.427	0.477	0.531	0.569	0.598	0.622	0.641	0.656	0.670	0.682	0.701
c. 40% Impervious	0.285	0.346	0.395	0.446	0.482	0.512	0.535	0.554	0.571	0.585	0.597	0.618
d. Pervious	0.125	0.185	0.230	0.277	0.312	0.330	0.362	0.382	0.399	0.414	0.429	0.454
2. Rectangle (length = 4 × width) concentrating in stated time												
a. Impervious	0.550	0.648	0.711	0.768	0.808	0.837	0.856	0.869	0.879	0.887	0.892	0.903
b. 50% Impervious	0.350	0.442	0.499	0.551	0.590	0.618	0.639	0.657	0.671	0.683	0.694	0.713
c. 30% Impervious	0.269	0.360	0.414	0.464	0.502	0.530	0.552	0.572	0.588	0.601	0.614	0.636
d. Pervious	0.149	0.236	0.287	0.334	0.371	0.398	0.422	0.445	0.463	0.479	0.495	0.522

Source: Manual on Sewerage & Sewage Treatment Plants, CPHEEO, 2013

4.4.1.6 Time of Concentration in storm drainage system (t_c)

The rainfall intensity 'i' in the rational formula is the average rainfall intensity over a given duration **equal to the time of concentration** for the drainage area. The rainfall intensity for the design storm can be obtained from the IDF relationship described in Chapter 3.

The time of concentration ($\boldsymbol{t_c}$) is defined as flow travel time taken from the hydraulically most remote point in the contributory catchment to the point under consideration**.** The time of concentration for conduit sizing is the time required for water to travel from the most hydraulically distant point in the total contributing catchment to the design point. Typically, this time consists of two components: (1) the time for surface flow to reach the first inlet i.e, t_0, and (2) the time to flow through the storm drainage system to the point of consideration i.e. t_f.

$$\mathrm{t_c} = \mathrm{t_0} + \mathrm{t_f} \tag{4.3}$$

The inlet time is dependent on the distance of farthest point in the drainage catchment to the inlet manhole as said above, as well as, on the shape, characteristic and topography of the catchment. It generally varies from 5 to 30 minutes in urban areas. In hilly areas the inlet time may be as low as 3 minutes, where steep slopes are encountered. However, the following formula is widely used to determine inlet time to reasonable accuracy.

4.4.1.6.1 Time of surface flow (t_0)

The formula to compute the time of surface flow has been developed by the Corps of Engineers (USA) from air field drainage data. The method was originally intended for use on airfield drainage problems, but has now been used frequently for surface flow in urban catchments. The formula to calculate time of surface flow (t_0) is given as follows:

$$t_o = \frac{0.994\ (1.1 - C) L^{0.5}}{S^{0.333}} \tag{4.4}$$

Where,
t_o =Time of surface flow (in minutes)
C = Rational Method runoff coefficient
L = Length of surface flow (m)
S = Surface Slope, in percentage (%)

Note: If slope (S) is expressed as a ratio, then the formula to be applied is

$$t_o = \frac{0.218\,(1.1 - C) L^{0.5}}{S^{0.333}} \tag{4.5}$$

4.4.1.6.2 Time of flow in conduit (t_f)

$$t_f = \frac{L_{Conduit}}{V} \tag{4.6}$$

The velocity of flow in m/s is computed from the Manning's equation

$$V = \frac{1}{n} R^{0.67} S^{0.5} \tag{4.7}$$

t_f : Time of travel in conduit, minutes
n : Manning's roughness coefficient
R : Hydraulic radius of conduit (m)
S : Longitudinal slope of conduit.

4.4.1.7 Partial Area Effect

In general, the appropriate time of concentration (tc) for calculation of the flow at any point is the longest time of travel to that point. However, in some situations, the maximum flow may occur when only part of the upstream catchment is contributing. Thus the product of a lesser C×A (Runoff coefficient × Catchment Area) and a higher I (resulting from a lower t_c) may produce a greater peak discharge than that if the whole upstream catchment is considered. This is known as the 'partial area effect'. This can occur in 2 cases as described below:

- The first case occurs when a highly impervious section exists at the most downstream area of a watershed and the total upstream area flows through the lower impervious area. When this occurs, two separate calculations should be made. First, calculate the runoff from the total drainage area with its weighted *C* value and the intensity associated with the longest time of concentration. Second, calculate the runoff using only the smaller impervious area. The typical procedure would be followed using the *C* value for the small impervious area and the intensity associated with the shorter time of concentration. Compare the results of these two calculations and use the largest value of discharge for design.

- The second case occurs when a smaller, impervious area is tributary to the larger primary watershed of less impervious area. When this occurs, two sets of calculations should also be made. First, calculate the runoff from the total drainage area with its weighted *C* value and the intensity associated with the longest time of concentration. Second, calculate the runoff to consider how much discharge from the larger primary area is contributing at the same time as the peak from the smaller, impervious tributary area. When the small area is discharging, some discharge from the larger primary area is also contributing to the total discharge. In this calculation, use the intensity associated with the time of concentration from the smaller, impervious area. The portion of the larger primary area to be considered is determined by this equation:

$$A_c = A\frac{t_{c1}}{t_{c2}} \tag{4.8}$$

Where,

A_c is the smaller, impervious tributary area to the larger drainage area
A is larger drainage area
t_{c1} is time of concentration of tributary area
t_{c2} is time of concentration of larger drainage area

Illustrative example is given in Appendix A 4.1.

4.4.2 Time Area Method

This method applies a convolution of the rainfall excess hyetograph with a time area diagram representing the progressive area contributions within a catchment in set time increments to generate runoff hydrograph of total flow to be routed through urban drain network. Computerized programmes are available such as TRR developed by UK Transport and Road Research Laboratory etc.

4.4.2.1 Travel Time

The excess rainfall over the catchment causes surface flow that passes through catchment channel to the point of catchment outlet. The time taken for surface flow from different points to the catchment outlet in the drainage catchment is called travel time. The time will be evidently more for remote points of the catchment and will be lesser for the points nearer to the catchment outlet. These points can be earmarked on the catchment from where the flow takes equal time to reach the catchment outlet. The line joining such points of equal time of travel is called isochrones. Different isochrones can be drawn expressing different time of flow and obviously the highest value of isochrones represents the time of concentration since it is the maximum time of flow from farthest point of the catchment.

Hydrographs are generated in time area method by convolution of the rainfall excess hyetograph with a time area graph generating progressive runoff contribution from sub catchments within the catchment in set time increments. To apply this method the catchment is first divided into a number of time zones separated by lines of equal travel time (isochrones) to outlet as shown in Figure 4.2.

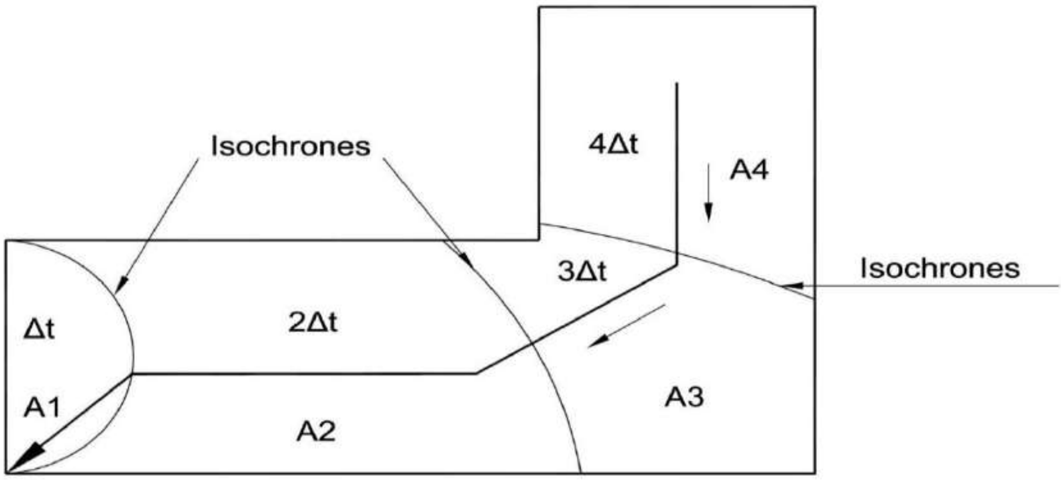


Figure 4.2: Isochrones

The ordinates of runoff hydrograph can be determined by applying each block of rainfall excess hyetograph given below in Figure 4.3 to the entire catchment.

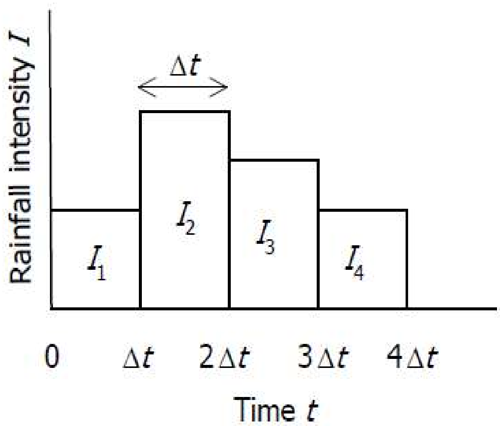


Figure 4.3: Hyetograph

The runoff from each sub area reaches the out fall at lagged intervals defined by the time area curve as shown below in Figure 4.4.

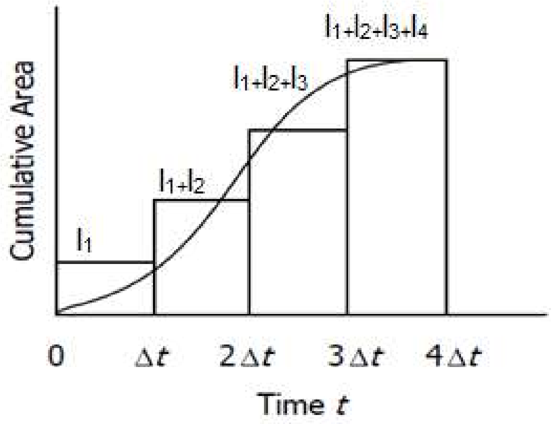


Figure 4.4: Time area curve

The simultaneous arrival of the runoff from areas A_1,A_2,A_3,A_4 caused by storm I_1,I_2,I_3,I_4 shall be determined by properly lagging and adding runoff contributions from sub catchments as explained below.

Travel time of each zone is Δt. Rainfall occurs over the entire catchment in time t. Now in first Δt interval, I_1 rainfall has fallen over the entire catchment and therefore after Δt interval, the discharge at outlet is contributed by sub-catchment A_1 from rainfall I_1.And hence, discharge $q_1 = A_1 * I_1$

Similarly I_2 rainfall has fallen in second Δt interval, the discharge A_2*I_1 and A_1*I_2 reach simultaneously at the outlet, hence discharge, $q_2 = A_2 * I_1 + A_1 * I_2$

Similarly by lagging and adding discharges $q_3 = A_3 * I_1 + A_2 * I_2 + A_1 * I_3$

$q_4 = A_4 * I_1 + A_3 * I_2 + A_2 * I_3 + A_1 * I_4$

After lapse of 4Δt the rain stops and rainfall generated by I_1 is entirely drained out. Rest of the incremental rainfalls falling over the sub catchment subsequently reach the outlet point as given by lagging and adding sub catchments flows here under.

$q_5 = A_4 * I_3 + A_3 * I_4 + A_2 * I_3$

$q_6 = A_4 * I_4 + A_3 * I_3$

$q_7 = 0$

A hydrograph can be developed by plotting discharges against time that can be used for designing drains/ conduits.

Illustrative example is given in Appendix A 4.2.

This method may be suitable for designing out fall drain for those catchments which are discharging at a given single point.

4.4.3 Unit Hydrograph Method

The unit hydrograph method is an outcome of investigation into the geometric properties of the surface runoff portion of the hydrograph in its relation to an effective rain that has fallen during a unit time. The unit hydrograph is, therefore, defined as the hydrograph of direct runoff resulting from an unit depth (1cm) of rainfall excess occurring uniformly over the catchment and at a uniform rate for a specified duration (D hours). A typical 30 min unit hydrograph is shown below in Figure 4.5.

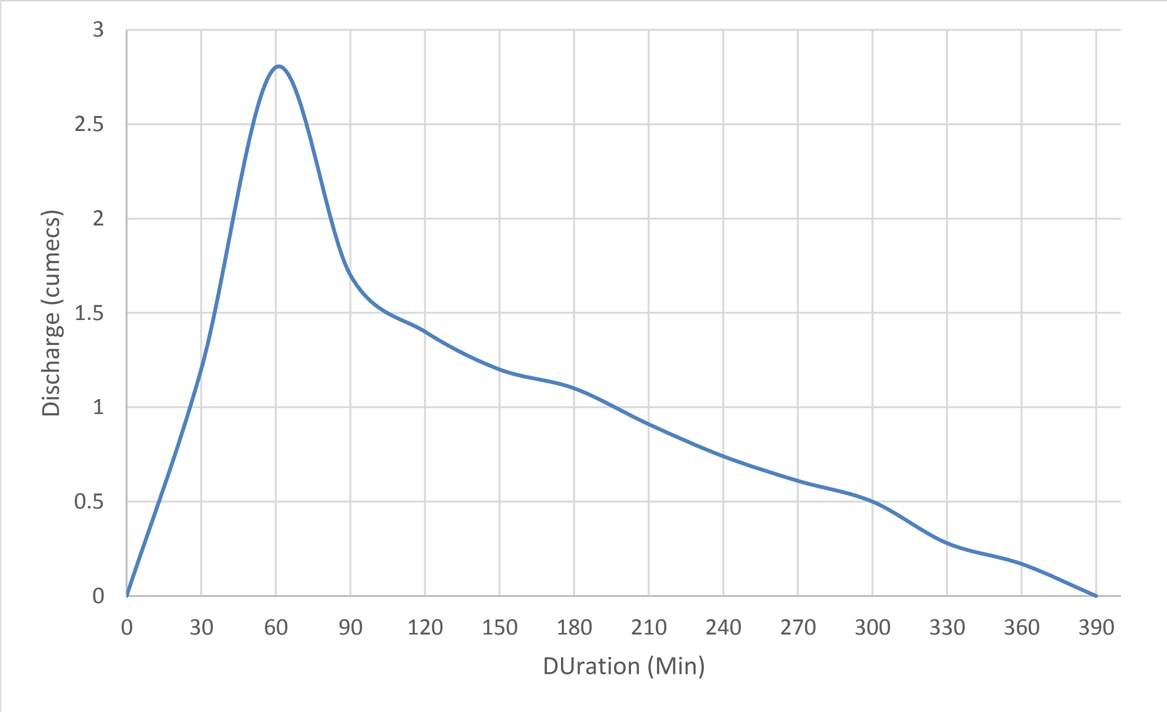


Figure 4.5: 30 min Unit Hydrograph

The unit hydrograph represents the lumped response of the catchment to a unit rainfall excess of D - hr duration to produce a direct runoff hydrograph to the rainfall excess. Hence the volume of water contained in the unit hydrograph must be equal to rainfall excess which is considered 1 cm over the entire given catchment.

If the rainfall excess in a duration D hours is r times the unit depth (1 cm), the ordinates of the resulting DRH will be r times the corresponding ordinates of the D hour unit hydrograph. Since the area under the D hour DRH should be r times the area under the corresponding D hour unit hydrograph, the base of the DRH will be the same as that of the unit hydrograph.

4.4.3.1 Limitations of Unit Hydrograph

- The upper limit of catchment area for use of Unit Hydrograph is prescribed not to be more than 5000 Sq Km whereas the lower limit of catchment area may not be less than 200 Ha;

- The catchment should not have large storages in terms of tanks, ponds, large flood bank storages etc., which affect the linear relationship between storage and discharge;

- If the precipitation is decidedly non-uniform unit hydrograph cannot be expected to give good results.

Illustrative example is given in Appendix A 4.3.

This method may be suitable for designing out fall drain for catchments of area not less than 200 hectare and discharging at a given single point. This may not be applicable for small urban catchments.

4.4.4 Rainfall- runoff process simulation

Following two methods for simulation of rainfall- runoff process are used for computation of storm runoff from urban watersheds.

(1) Kinematic wave equation is applied to describe the overland flow on the watershed considered as a wide plane with very shallow depth of flow which is technically termed as sheet flow. For a given rate of rainfall and infiltration varying discharges from unit width of the watershed can be evaluated and adding discharges of all such unit widths, total discharge varying with each time step can be computed in shape of hydrograph at the outlet of the watershed. The Saint Venant equations describe the one dimensional unsteady flow which is applicable in this case. In kinematic wave motion inertial and pressure forces have negligible effects, therefore continuity equation given in (4.9) and Manning equation given in (2) are combined as given in (4.10) which is used to simulate and compute the runoff from the watershed:

$$\frac{\partial y_o}{\partial \mathrm{t}} + \frac{\partial q_o}{\partial \mathrm{X}} = (i - f) \tag{4.9}$$

$$q_o = \mu_o \, (Y_o)^\wedge \, m_o \tag{4.10}$$

Where,

q_o : Variable flow per unit width of overland flow plane
μ_o : (1/N* $S_o^{0.5}$)
m_o : 5/3
S_o : Average slope of overland flow
y_o : Mean depth of out flow
$(i - f)$: Rate of excess rainfall (rainfall – infiltration)
t : Time
x : Spatial coordinate
N : Manning roughness coefficient (Values may be seen in Appendix A 5.8)

Combining equation 4.9 & 4.10, complete kinematic wave equation is obtained as follows

$$\frac{\partial Y_o}{\partial \mathrm{t}} + \mu_o\, m_o\, y_o{}^{(m_o-1)}\frac{\partial y_o}{\partial \mathrm{X}} = (i - f) \qquad (4.11)$$

In the application of above formulae the lateral flow is considered equal to difference between the rates of rainfall and infiltration and the overland flow is taken to be flow per unit width of the plane. The equations (3) has one dependable variable so that it can be solved to give a relationship for y_o in terms of x,t and excess rainfall depth $(i - f)$. Once y_o is found, it can be substituted back into equation (2) to obtain the value of q_o . The solution of equation (3) can be worked out by finite difference approximations. Nevertheless, it is easier to solve the equation by computer software to develop the runoff hydrograph at the outlet of the watershed.
Hydrologic Engineering Center, US Corps of Engineers research facility in Davis, California has developed such a computer program named HEC-1 that is widely used to develop runoff hydrograph from the watershed at its outlet.

(2) Non Linear reservoir method

Nonlinear reservoir method for rainfall runoff simulation can also be used to compute runoff quantity for single event or long term simulation primarily from urban catchments as per governing equations discussed below.

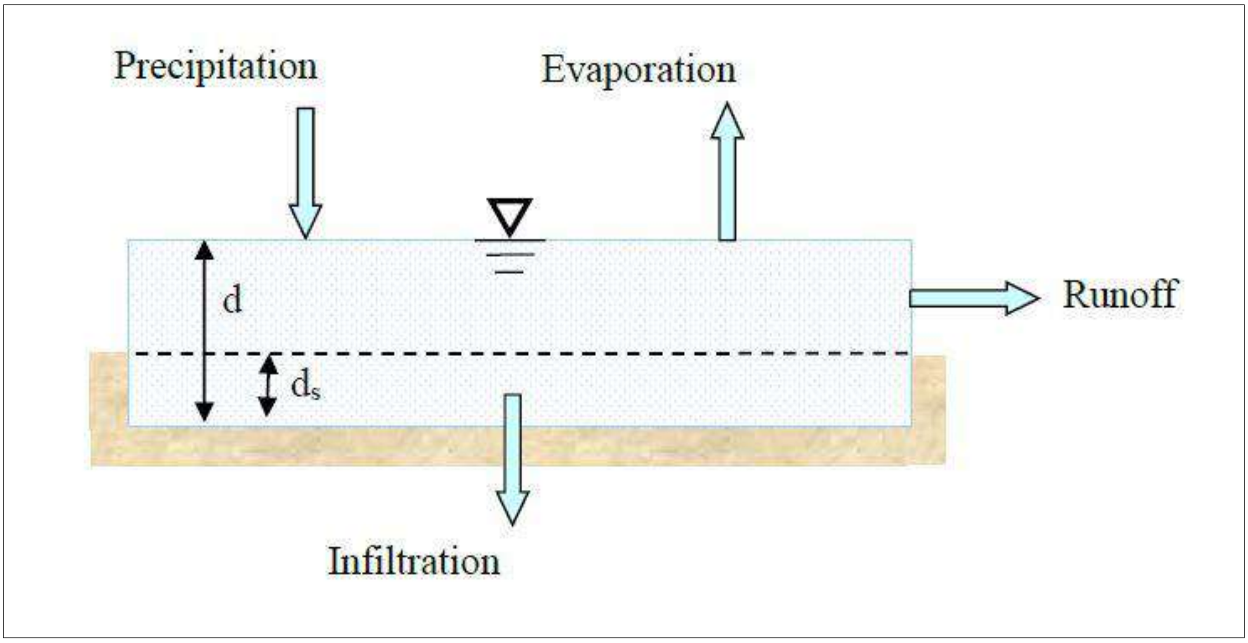


Figure 4.6: Nonlinear reservoir model

From the above Figure 4.6, the sub catchment experiences inflow from precipitation and losses from evaporation and infiltration. The net excess ponds atop the sub catchment surface to a depth d. Ponded water above depression storage depth d_s can become runoff outflow q. Depression storage accounts for initial rainfall abstraction. From conservation of mass, the net change in depth d per unit time is the difference between inflow and out flow rates over the catchment i.e.

$$\frac{\partial \mathrm{d}}{\partial \mathrm{t}} = i - e - f - q \qquad (4.12)$$

Where,
i= rate of rainfall
e=surface evaporation rate
f=infiltration rate
q= runoff rate
i, e ,f and q are expressed as flow rate per unit area.

Assuming that flow across the sub catchment surface behaves as it were a uniform flow within a rectangular channel of width w, height d-d_s, and slope s. The Manning's equation can be used to express the runoff's volumetric flow Q as

$$Q = \frac{1}{N} A\, R^{2/3}\, {S_o}^{1/2} \qquad (4.13)$$

Where,
n= Manning's roughness coefficient
S= average slope of the catchment
A_x= area across the sub catchment width through which the runoff flows.

Referring to the figure 4.6, A_x is the rectangular area with width w and height d-d_S. Because w will always be much larger than d, it follows that $A_X = W * (\mathrm{d} - d_s)$ and $R_X = (\mathrm{d} - d_s)$

Substituting the values of R_x and A_x in equation 4.13:

$$Q = \frac{(w * s^{\frac{1}{2}})}{N} * (\mathrm{d} - d_s)^{\frac{5}{3}} \tag{4.14}$$

To obtain the runoff per unit area,

$$q = \frac{(w * s^{\frac{1}{2}})}{AN} * (\mathrm{d} - d_s)^{\frac{5}{3}} \tag{4.15}$$

Substituting this equation into the original mass balance relation results given in equation 4.12:

$$\frac{\partial \mathrm{d}}{\partial \mathrm{t}} = i - e - f - \mu (\mathrm{d} - d_s)^{\frac{5}{3}} \tag{4.16}$$

Where,

$$\mu = \frac{(w * s^{\frac{1}{2}})}{AN}$$

The above equation is a nonlinear differential equation for known values of i, e, f, d_s and µ it can be solved numerically over each time step for ponded depth by numerical integration method and subsequently value of runoff Q that can be developed in shape of runoff hydrograph at the outlet of the watershed.

Environmental Protection Agency (US) has developed computer software named “SWMM” which is widely used to compute and develop the runoff hydrograph at the outlet of the watershed.

4.5 Climate Change effects on Urban Storm Runoff

Based on intensive research across the globe as well as those reported through IPCC, it has been established that global warming induced climate change is causing change in rainfall precipitation pattern. Various studies in India including those by IMD also strengthen above changing pattern. It is established that rise in atmospheric temperature lead to intensifying Earth Hydrologic Cycle causing short duration heavy intensity precipitations. Each 1 deg C rise in atmospheric temperature leads to 7 % increase in water vapor in the atmosphere. Countries like UK have already recommended an increase of 20 % in the design storm runoff to account for change in rainfall pattern due to climate change.

However, in the large country like India, the projected percentage change in rainfall intensity may be carried out after detailed analysis of past 30 years or more data before incorporating in the design. This is essential due to the fact that in IMD study many rainfall stations have also recorded reduction in rainfall intensity.

Firstly, to account for impact of climate change on rainfall and consequent change in design discharge, Intensity – Duration – Frequency curves needs to be periodically updated for a given catchment or locality intending to design new or retrofitting old storm water drains.

Additionally, increased volume of runoff from higher intensity of rainfall on account of climate change can also be channelized by implementing BMPs, LIDs etc. as recommended in Chapter 10.

CHAPTER 5: HYDRAULIC DESIGN OF STORM WATER DRAINS

5.1 General

The Chapter gives broad coverage of theory and practice of open channel hydraulic in planning and design of storm water channels and conduits that convey storm runoff resulting from rainfall over urban catchments as described in Chapter 3 & Chapter 4 of this Manual. Flow equations to compute the hydraulic parameters required in designing the channels, gutters and conduits under different flow conditions are given with illustrative examples to show the application of the flow formulae. A brief outline about design aspects of the engineered natural channels have been discussed in the final section of this chapter.

5.2 Storm water flows in Channels and Conduits

Storm water flows in channels/ conduits in contact with atmospheric air is said to be an open channel flow or free surface flow.

If the rate of discharge remains constant with time at a given cross section, the flow is said to be steady and if it varies with time, then the flow is called unsteady flow.

If the velocity and depth of flow are the same at every section of channel / conduit, the steady open channel flow is said to be uniform flow and if the velocity, depth or both are changing then the flow is known as non-uniform flow.

When fluid flows in a parallel direction without interruption between each layer, it is defined as laminar flow and if it moves in irregular paths, it is said to be turbulent flow.

Storm water flows in open channel / conduit are under unsteady and turbulent flow conditions but to simplify the design process, it is assumed to flow in steady turbulent conditions either uniform, non-uniform gradually or rapidly varied flow.

Flow Formulae which are applicable in design computations of storm channels/ conduits.

1. Reynolds's number

For open channel flow of any cross section:

$R_e = VR / \nu$ (5.1)

For flow in pipe of diameter D

$R_e = VD / \nu$ (5.2)

Where,
R_e = Reynold's Number (Dimensionless)
V = Cross- sectional mean velocity in m/sec
D = Diameter of pipe
R = Hydraulic Radius (A/P) in m
ν = Kinematic Viscosity in m^2/sec
P = wetted perimeter in m

R_e gives a dimensionless number which is called Reynold's number. It determines whether the flow is laminar or turbulent. It is found that in open-channel, flow is laminar when $R_e \leq 500$ and turbulent when $R_e > 1000$, whereas in pipe flow, the flow is laminar when, $R_e \leq 2000$ and flow is turbulent when $R_e > 4000$.

2. **Specific Energy E_s**: It is defined as energy of the flow with reference to the channel bed as the datum.

$$E_s = y + V^2/2g \tag{5.3}$$

Where,
y = Depth of water in the channel
V = Mean cross section velocity
g = Acceleration due to gravity
E_s = Specific energy

3. **Froude number**

$$F_r = V / (gD_m)^{1/2} \tag{5.4}$$

Where,
F_r = Froude number (Dimensionless)
V = Mean velocity in m/sec
D_m = Hydraulic mean depth in m (cross section area of flow/width of the channel)
g = acceleration due to gravity in m/sec^2

4. **Critical flow:** The general equation for Critical flow:

$$Q^2B/gA^3 = 1 \tag{5.5}$$

Where,
Q = Discharge
B = Width of water surface in the channel
A = Cross section area of water flow
g = Gravity

For a fixed discharge, the specific energy is minimum at critical depth.

For all other values of specific energy there are two alternate depths, one is subcritical depth when flow depth is greater than critical depth and other is super critical depth when flow depth is less than critical depth.

In steady Uniform flow, the flow depth is known as normal depth. The slope at normal depth is said to be mild slope, at critical depth the slope is said to be critical slope and at super critical depth the slope is known as steep slope.

5. Manning's Equation

$$V = (1/n) \times R^{2/3} \times S^{1/2} \tag{5.6}$$

Where,
V = Velocity of flow in m/sec
R = Hydraulic radius (flow area/ wetted perimeter) in m.
S = Hydraulic slope in m/m
n = Manning's coefficient of roughness
P= wetted perimeter in m
A= Area of cross section of water area in m^2
Q= Discharge in m^3/sec

Manning Equation for uniform flow in open channels, in terms of discharge can be written as,

$$Q = (1/n) \times (A^{5/3}/P^{2/3}) \times S^{1/2} \tag{5.7}$$

Owing to its simplicity and acceptable degree of accuracy in a variety of practical application Manning's formula valid for turbulent flow which is the most widely used in uniform flow formula for designing storm water pipe drains and channels. Due to its long practical use, values of n for a very wide range of channels are available as given in the Table 5.1. Manning formula is valid for fully turbulent flow.

While choosing the storm water pipe diameters, minimum required diameter is computed and the next larger commercial available pipe diameter is selected. Once the value of peak discharge (Q) from rational method or by other runoff estimation method is computed, the minimum pipe diameter can be calculated by the Manning equation as given below:

For circular section: Area of cross section = A = $\pi D^2/4$ and Hydraulic Radius, R=D/4

Therefore,

$$Q = (0.3107/n) \times D^{8/3} \times S^{1/2} \quad (5.8)$$

Where,
D = Diameter of pipe in m
Q = Discharge for full section in m^3/sec (Peak discharge calculated by rational method as described in chapter 4)

In circular conduits maximum velocity occurs at 0.81 depth and maximum discharge occurs at 0.95 depth.

To keep maximum discharge, it is recommended to design the storm channel/ conduit at 0.81 depth of the channel conduit. Check that Froude no may not exceed 0.8 in order to maintain sub critical velocity.

Table 5.1: Coefficient of roughness for channel flow for use in manning's formula

Type of Material	Condition	Manning's n
Salt gazed stone ware pipe	(a) Good	0.012
	(b) Fair	0.015
Cement concrete pipes (With collar joints)	(a) Good	0.013
	(b) Fair	0.015
*Spun concrete pipes (RCC & PSC) with S / S Joints (Design value)		0.011
Masonry	Neat Cement Plaster	0.018
	Sand and Cement Plaster	0.015
	Concrete, steel troweled	0.014
	Concrete, wood troweled	0.015
	Brick in good condition	0.015
	Brick in rough condition	0.017
	Masonry in bad condition	0.020
Stone work	Smooth, dressed ashlar	0.015
	Rubble set in cement	0.017
	Fine, well packed gravel	0.020
Earth	Regular surface in good condition	0.020
	In ordinary condition	0.025
	With stones and weeds	0.030
	In poor condition	0.035
	Partially obstructed with debris or weeds	0.050
Steel	Welded	0.013
	Riveted	0.017
	Slightly tuberculated	0.020
	With spun cement mortar lining	0.011
Cast Iron / Ductile iron	Unlined	0.013
	With spun cement mortar lining	0.011
Asbestos Cement		0.011

Type of Material	Condition	Manning's n
Plastic (Smooth)		0.011
FRP		0.01
HDPE / UPVC		0.01

Note: Values of n may be taken as 0.015 for unlined metallic pipes and 0.011 for plastic and other smooth pipes
* n value for Spun concrete pipes (RCC & PSC) with S / S Joints may be taken as given by the manufacturer.

Source: Manual on Sewerage and Sewage Treatment Systems, by CPHEEO, 2013

5.3 Design consideration for surface/ sub surface drains

Sub critical flow is normally maintained by keeping Froude number not exceeding 0.8.Critical flow condition develops when Froude no equals 1.0. In open channel flow design, critical state of flow should be avoided as under such condition the water surface becomes unstable and wavy. It is, therefore, recommended that the channel flow should be designed so that the Froude no should not exceed 0.8 preferably and self-cleansing velocity as recommended in Table 5.2.

5.3.1 Permissible limit of Velocity in storm conduits

To ensure that deposition of suspended solids does not take place, self-cleansing velocities using Shield's formula is considered in the design of channels / conduits.

From findings of Shields, Camp derived the formula:

$$\boldsymbol{V = \frac{1}{n} * R^{\frac{1}{6}}\left[k_s(S_s - 1)d_p\right]^{(1/2)}} \tag{5.9}$$

Where,
n = Manning's n
R = Hydraulic Mean Radius in m
k_s = Dimensionless constant with a value of about 0.04 to start motion of granular particles and 0.8 for adequate self-cleansing of conduits
S_s= Specific gravity of particles
d_p = Particle size in mm

Shields formula indicates that velocity required to transport material in conduits is only slightly dependent on conduit shape and depth of flow but mainly dependent on the particle size and specific weight. A velocity of 0.6 mps would be required to transport sand particles of 0.09 mm with a specific gravity of 2.65 that are commonly found in storm water from urban catchments.

Table 5.2: Design velocities to be ensured in gravity storm conduits

S. No.	Criteria	Value
1	Minimum velocity	0.6 m/s
2	Maximum Velocity	3 m/s

Source: WPCF, ASCE, 1982

Note:

i. For hilly regions, maximum velocity to be permitted in storm water conduits should be 6.0 m/s for plastic pipes or other pipes lined with plastics.

5.3.2 Freeboard in channel

The freeboard is the vertical distance from the water surface of designed flow condition to the top of the channel. The importance of this factor depends on the consequence of overflow of the channel bank. Freeboard should be sufficient to prevent waves, super elevation changes, or fluctuations in water surface from overflowing the sides. Recommended value of minimum freeboard for different discharges is as given in Table 5.3.

Table 5.3: Minimum Free Board for channels

	Drain Size	Free Board
(i)	Beyond 300 mm bed width	10 cm
(ii)	Beyond 300 mm & up to 900 mm bed width	15 cm
(iii)	Beyond 900 mm & up to 1500 mm bed width	30 cm

For larger drains, the free board shall be higher up to 90 cm depending upon the discharge. For storm channel free board is not defined as the storm water conduits are supposed to run full.

Source: IRC SP 50 – 2013

However, a steep gradient channel (where Normal depth is less than critical depth) should have a freeboard height equal to the flow depth to compensate for the large variations in flow caused by waves, splashing, and surging.

5.3.3 Curves / bends in drains

Curves and bends are sometimes unavoidable in drain alignments. The complexity in design arises due to increase in friction losses along the curve that causes serious local erosion due to spiral flow motion induced by the centrifugal force which is very pronounced and irregular in the bend.

Therefore, in order to reduce the super elevation of the water surface that occurs due to the difference in elevation of water surface between inside and outside wall of the bend at the same section and maintain the freeboard, a minimum radius of curvature of 3 times the width of the drain should be provided in horizontal curve.

Benching should be provided at the bend to minimize the sedimentation at the inner side of the bend.

5.3.4 Junction Sump for storm water drain intersection

A sump of sufficient size shall be provided where drains converge or intersect. The minimum internal width of the sump shall not be less than 2 times the width of the drain leading away from the sump. Drains shall enter the sump at angles less than a right angle and at different levels wherever possible. The invert level of the downstream drain shall be lower than the invert level of the sump so that no stagnant water will collect in the sump.

5.3.5 Hydraulically Efficient Channel Section

The conveyance of a channel section of a given area increases with a decrease in its perimeter. Hence a channel section having the minimum perimeter for a given area of flow provides the maximum value of the conveyance. With the slope, roughness coefficient and area of flow fixed, a minimum perimeter section will represent the hydraulically efficient section as it conveys the maximum discharge. This channel section is also called the best section.

Table 5.4: Proportions of Some Most Efficient Sections

Sl. No.	Channel Shape	Area (A_e)	Wetted Perimeter (P_e)	Width (B_e)	Hydraulic Radius (R_e)	Top width (T_e)	$\frac{Q_n}{y_e^{8/3} S_0^{1/2}} = K_e$
1.	Rectangle (Half square)	$2Y_e^2$	$4Y_e$	$2Y_e$	$\frac{Y_e}{2}$	$2Y_e$	1.260
2.	Trapezoidal (Half regular hexagon, $m = \frac{1}{\sqrt{3}}$)	$\sqrt{3}Y_e^2$	$2\sqrt{3}\ Y_e$	$\frac{2}{\sqrt{3}}\ Y_e$	$\frac{Y_e}{2}$	$\frac{4Y_{ec}}{\sqrt{3}}$	1.091
3.	Circular (semi-circular)	$\frac{\pi}{2}Y_e^2$	πY_e	$D = 2Y_e$	$\frac{Y_e}{2}$	$2Y_e$	0.9895
4.	Triangle (Vertex angle-90º)	Y_e^2	$2\sqrt{3}\ Y_e$	-	$\frac{Y_e}{2\sqrt{2}}$	$2Y_e$	0.500

Where,

- e subscript denotes most efficient
- Y_e is the depth of flow for most efficient section

- Q_n is discharge
- S_o is bed slope

Source: Flow in open channels by K. Subramanaya

Illustrative Example 5.1:

Design the most efficient trapezoidal section for the following design parameters:

- Discharge (Q) = 20 m^3/s
- Bed slope (S) = less than 0.0003
- Manning (n) = 0.013
- Depth of flow (y) = Less than 3.0 m

Solution:

As per Table 5.4, conditions of most efficient Trapezoidal section:

$$\frac{Q_n}{y^{8/3} \times S^{1/2}} = 1.091$$

So, $y = \left(\frac{1.091 \times 0.1414}{20 \times 0.013}\right)^{\frac{3}{8}} = 2.88 \text{ m}$

As, y is less than 3.0 m; Hence, its OK

As per Table 5.4, Hydraulic radius $= \frac{y}{2}$

Hydraulic radius $= \frac{2.88}{2} = 1.44\ m$

As per Manning's formula, v = 1/n × $R^{0.66}$ × $s^{0.5}$

v $= \frac{1}{0.013} \times 1.44^{0.66} \times 0.0002^{0.5} = 1.387$ m/s

As per Table 5.4, v is greater than 0.6 m/s and less than 3 m/s. Hence, it is self-cleansing velocity and acceptable value of velocity.

Illustrative Example: 5.2

An open triangular channel of V shaped with each side inclined at 45° to vertical. It carries a discharge of 40 LPS. When the depth of flow at the centre is 225 mm. Calculate the slope of the channel.

Solution:

Given Data

- Discharge (Q) = 40 LPS
- Manning's constant (n) = 0.013

- $\angle\theta = 45^{\circ}$
- Depth of flow (y) = 0.225 m.

As per Table 5.4, condition for best hydraulic section:

$$\frac{Qn}{y^{8/3} \times S^{1/2}} = 0.5$$

$$\frac{0.04 \times 0.013}{0.225^{8/3} \times S^{1/2}} = 0.5$$

Solving the Equation: S = 0.0030231

Illustrative Example: 5.3

Find the most efficient section of rectangular channel to carry 300 lps when the bed slope is 1 in 1000. (Given n as 0.013)

Solution:

Given data

- Discharge (Q) = 300 lps
- Bed slope is (S) 1:1000
- Manning Constant (n) = 0.013

As per Table 5.4, condition for most efficient rectangular channel:

$$\frac{Qn}{y^{8/3} \times S^{1/2}} = 1.260$$

Solving the equation, y = 0.418 m

As per the Table 5.4:

- Area of cross-section = $2y^2 = 2 \times 0.418^2 = 0.349\ m^2$
- Width of channel = 2 y = 2 × 0.418 = 0.836 m
- Hydraulic radius, R = y/2 = 0.418/2 = 0.209 m
- Velocity of flow is (V) = $1/n \times R^{0.66} \times s^{0.5} = 1/0.013 \times 0.209^{0.66} \times 0.001^{0.5}$ = 0.865 m/s

5.3.6 Partially Filled Circular Section

The elements shown in figure 5.1, those of area and hydraulic radius are static or elements of shape and those of roughness, velocity and discharge are dynamic elements of flow. The basis for computation of both group of elements are shown below:

$$\frac{a}{A} = \frac{\theta}{360^\circ} - \frac{\sin\theta}{2\pi} \qquad (5.10)$$

$$\frac{r}{R} = 1 - \frac{360^\circ \sin\theta}{2\pi\theta} \qquad (5.11)$$

$$\frac{v}{V} = \left(\frac{r}{R}\right)^{2/3} \text{, where n is Constant} \qquad (5.12)$$

$$\frac{q}{Q} = \frac{a}{A}\left(\frac{r}{R}\right)^{2/3} \text{, where n is constant} \qquad (5.13)$$

Where,

A= cross section of the circular section,
a = cross section of the partially filled circular section
R = hydraulic radius of the full circular section,
r = hydraulic radius of partially filled section
V = velocity of flow of full section
v =velocity of flow of partially filled section
Q = discharge from full section flow
q = discharge from partially filled section

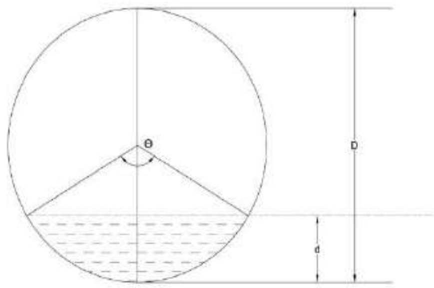


Figure 5.1: Partially Filled Circular Section

From above equation ratios of $\frac{d}{D}, \frac{v}{V}, \frac{a}{A}, \frac{q}{Q}$ can be calculated and tabulated and values in between can also be interpolated. The table 5.5 and graphical presentation of these ratios for constant n and variable n are given in figure 5.2 and figure 5.3:

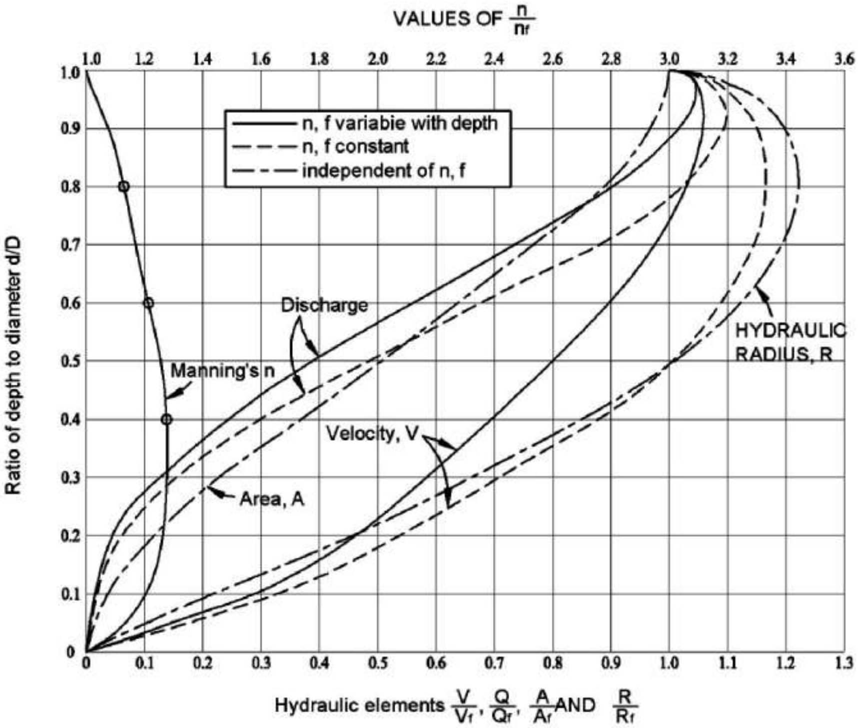


Figure 5.2: Hydraulic – Element graph for circular storm water conduits

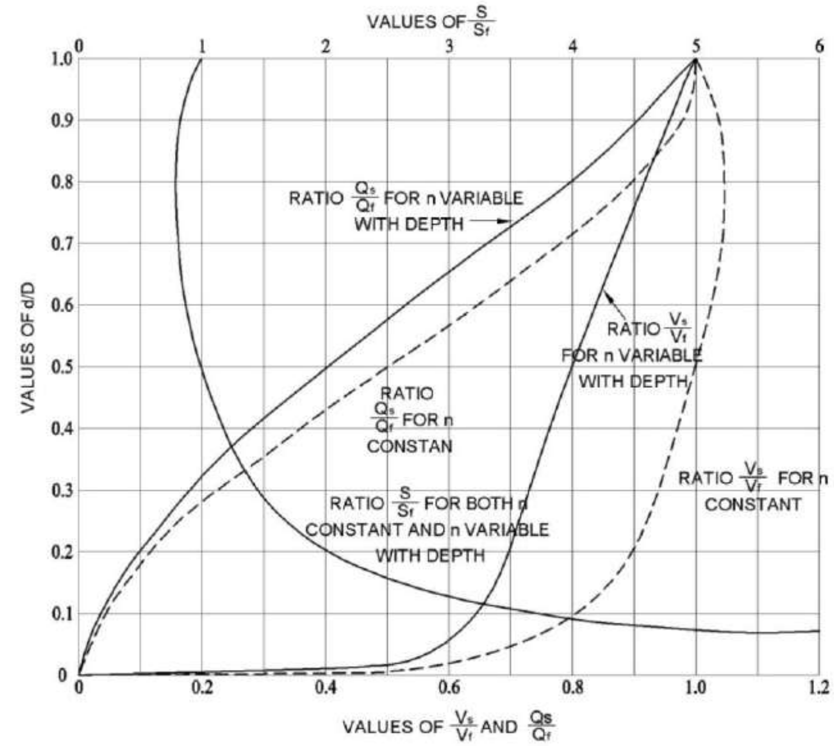


Figure 5.3: Hydraulic elements of circular storm water conduits that possess equal self-cleansing properties at all depths

Table 5.5: Hydraulic Properties of circular sections for manning's formula

Constant (n)			Variable (n)		
d/D	**v/V**	**q/Q**	**n_o/n**	**v/V**	**q/Q**
1.0	1.000	1.000	1.00	1.000	1.000
0.9	1.124	1.066	1.07	1.056	1.020
0.8	1.140	0.968	1.14	1.003	0.890
0.7	1.120	0.838	1.18	0.952	0.712
0.6	1.072	0.671	1.21	0.890	0.557
0.5	1.000	0.500	1.24	0.810	0.405
0.4	0.902	0.337	1.27	0.713	0.266
0.3	0.776	0.196	1.28	0.605	0.153
0.2	0.615	0.088	1.27	0.486	0.070
0.1	0.401	0.021	1.22	0.329	0.017

Where,
D = Full Depth of Flow (Internal dia)
d = Actual Depth of Flow
V = Velocity at full depth
v = Velocity at depth 'd' ’
Q = Discharge at full depth
q = Discharge at depth 'd'

Illustrative Example 5.4

A 225 mm dia storm water drain is to discharge of 0.005 cumec at a required gradient of 1 in 1500. Find out the depth and velocity of flow in storm drain. Assume Manning's coefficient 'n' as 0.013.

Solution:

Using Manning's formula, discharge through the pipe while flowing full is given by;

$$Q = \frac{1}{n} A. R^{2/3}. \sqrt{S}$$

$$Q = \frac{1}{0.013} . \frac{\pi}{4} (0.225)^2 . (\frac{0.225}{4})^{2/3} . \sqrt{\frac{1}{1500}}$$

Hence, Q = 0.0116 m^3/s,
Now, $V = \frac{0.0116}{\frac{\pi}{4}\times(.225)^2}$ = 0.292 m/s
Discharge through the Drain when flowing partially full (q) = 0.005 cumec when

$\frac{q}{Q} = \frac{0.005}{.0116} = 0.431$, then from above table, $\frac{d}{D} = 0.458,$ $\frac{v}{V} = 0.959$

Depth & Velocity of partially filled drain,

$\frac{d}{D} = 0.458,$ d = 0.458*.225 = 0.103 m

$\frac{v}{V} = 0.959,$ v = 0.959*.292 = 0.28 m/s

5.4 Design Sheet

The designer should tabulate the complete hydraulic design of drains and conduits for entire given network of project catchment area in the relevant columns given in Table 5.6 and Table 5.7.

Table 5.6: Computation sheet for Storm Water Conduit

Drain No	Location of Drain			Tributary Area (hectares) Increment				t_c time of concentration (min)			Intensity of rainfall (mm) (I)	Runoff Coeff. "C"	Runoff (Q) (m^3/hr) 10CIA	Flow Q Lps		Design						Profile				
	Street	Manhole from	Manhole to	0.7 Imp factor	0.2 Imp factor	Eq 100 % Imp factor	Total area	Time of (t_o) inlet	Time of flow in drain t_f	Total $t_c = t_o + t_f$					Dia (mm)	Slope I in	Capacity lps	Velocity mps		Length m	Fall m	Drop in Manhole	Ground elevation		Invert Elevation	
																		Full flow Vf	Actual velocity Va				Upper end	Lower end	Upper end	Lower end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27

Prepare a working plan layout and profile of the storm drainage system establishing the following design information:

Columns 1 to 4 identify the location of drain, street and manholes

Columns 5 to 6 record the increment in tributary area with the given imperviousness factors

Column 7 gives the tributary area increment with equivalent 100 % imperviousness factor

Column 8 records the total area served by each drain

Column 9 records the time of inlet (t_o) at each upper end of the line (drain) from the formula given in Chapter 4 clause 4.4.1.6.

Column 10 records the time of flow (t_f) in each drain from the formula given in Chapter 4 clause 4.4.1.7.

Column 11 is the total time of concentration (t_c) for each drain.

Column 12 is the value of intensity of rainfall in mm corresponding to the time of concentration from IDF curve for required return period

Column 13 is the runoff coefficient from the Table 4.3 given in Chapter 4

Column 14 is the value of runoff (CIA) in m^3/hr from each tributary area from the Rational formula given in Chapter 4.

Column 15 is the value of runoff converted in lps from each tributary area

Column 16 – 20 records the chosen size, required grade resulting capacity, full and actual velocity of flow for each drain or line. These designs of storm water conduit are computed from the Manning's equation for each required flow and maintaining a self-cleansing velocity.

Column 21 – 27 identifies the profile of the drain

Column 21 is taken from the plan

Column 22 is Column 21 × Column 17

Column 23 is the required drop in manholes is obtained directly from the recommended values in Chapter 11, section 11.3.14.5 Drop in Manhole'

Column 24 & 25 is upper and lower end Ground elevation

Column 26 & 27 gives invert elevation at the upper end with a minimum cover of 0.6 m at starting manhole. In case a manhole having more than one inlet, the drop in the manhole is considered with respect to the lowest invert level of the inlets to fix the invert level of the outlet.

Table 5.7: Computation sheet for Storm Water Drain

Drain No	Location of Drain			Tributary Area (hectares) Increment				t_c time of concentration (min)			Intensity of rainfall (mm) (I)	Runoff Coeff. “C”	Runoff (Q) (m^3/ hr) 10CIA	Flow Q Lps		Design						Profile			
	Street	from Start Node	to Stop Node	0.7 Imp factor	0.2 Imp factor	Eq 100 % Imp factor	Total area	Time of (t_o) inlet	Time of flow in drain t_f	Total $t_c = t_o + t_f$					Section of the drain	Slope m / 1000	Capacity lps	Velocity mps: Full flow Vf	Velocity mps: Actual velocity Va	Length m	Fall (m	Ground elevation: Upper end	Ground elevation: Below end	Invert Elevation: Upper end	Invert Elevation: Below end
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26

Prepare a working plan layout and profile of the storm drainage system establishing the following design information:

Columns 1 to 4 identify the location of drain, street and manholes

Columns 5 to 6 record the increment in tributary area with the given imperviousness factors

Column 7 gives the tributary area increment with equivalent 100 % imperviousness factor

Column 8 records the total area served by each drain

Column 9 records the time of inlet (t_o) at each upper end of the line (drain) from the formula given in Chapter 4 clause 4.4.1.6.

Column 10 records the time of flow (t_f) in each drain from the formula given in Chapter 4 clause 4.4.1.7.

Column 11 is the total time of concentration (t_c) for each drain.

Column 12 is the value of intensity of rainfall in mm corresponding to the time of concentration from IDF curve for required return period

Column 13 is the runoff coefficient from the Table 4.3 given in Chapter 4

Column 14 is the value of runoff (CIA) in m^3/hr from each tributary area

Column 15 is the value of runoff converted in lps from each tributary area

Column 16 – 20 records the chosen size, required grade resulting capacity, velocity of flow for each drain or line. These designs of storm water conduit are computed from the Manning's equation for each required flow and maintaining a minimum velocity

Column 21 – 26 identifies the profile of the drain

Column 21 is taken from the plan

Column 22 is Column 21 × Column 17

Column 23 & 24 is upper and lower end Ground elevation

Column 25 & 26 gives invert elevation at the upper end with a minimum cover of 0.6 m at starting manhole. In case a manhole having more than one inlet, the drop in the manhole is considered with respect to the lowest invert level of the inlets to fix the invert level of the outlet.

5.5 Gutters and Inlets

5.5.1 Gutter

A pavement gutter is defined as a section of pavement adjacent to the roadway which conveys water during a storm runoff event. It may include a portion or all of a travel lane. Gutter sections usually have a triangular shape with the curb forming the near-vertical leg of the triangle. Conventional gutters may have a straight cross slope or a composite cross slope where the gutter slope varies from the pavement cross slope.

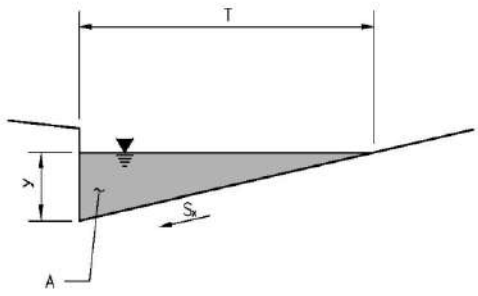


Figure 5.4 : Gutter section with uniform cross slope

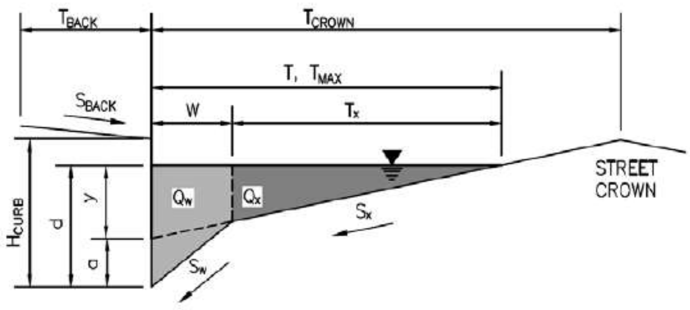


Figure 5.5 :Typical gutter section—composite cross slope

5.5.2 Design of Gutters

Gutter Flow calculations are necessary to establish the spread of water on the shoulder, parking lane, or pavement section. Streets with uniform cross slopes like that shown in Figure 5.4 are found in urban areas. Since gutter flow is assumed to be uniform for design purposes, Manning's equation is appropriate with a slight modification to account for the effects of a small hydraulic depth (A/T).

For a triangular cross section as shown in Figure 5.4, Manning's equation for gutter flow is written as:

$$\mathbf{Q = \frac{K_c}{n} S_x^{5/3} S_L^{1/2} T^{8/3}} \qquad \textbf{(5.14)}$$

Where,
K_c = empirical constant equal to 0.376

n : Manning's roughness coefficient for gutter flow as given in Table 5.8
Q : Flow rate (m^3/s)
T : Width of flow (spread), m
S_x : Cross slope, m/m
S_L : Longitudinal slope, m/m
Equation neglects the resistance of the curb face since this resistance is negligible.

The flow depth (m) can be found using:
$y = TS_x$ (5.15)
And,
The cross-sectional flow area, $A = \frac{S_x T^2}{2}$ (5.16)

Table 5.8: Manning's n for gutter flow

Surface type	n
Concrete	0.013
Hot mix asphaltic concrete	0.015
Sprayed seal	0.018

Illustrative Example 5.5:

A triangular gutter of concrete has a longitudinal slope of 1%, cross slope of 2%, and a curb depth of 0.2 m. Determine the flow rate and flow depth if the spread is limited to 2 m.

Solution:

$$\mathbf{Q = \frac{K_c}{n} S_x^{5/3} S_L^{1/2} T^{8/3}}$$

Where,

K_c = empirical constant = 0.376
n : Manning's roughness coefficient = 0.012
Q : Flow rate (m^3/s)
T : Width of flow (spread) = 2 m
S_x : Cross slope = 0.02
S_L : Longitudinal slope = 0.01

$$Q = \frac{0.376}{0.012} \times 0.02^{\frac{5}{3}} \times 0.01^{\frac{1}{2}} \times 2^{\frac{8}{3}} = 31.33 \times 0.00145 \times 0.1 \times 6.364 = 0.029 \ m^3/s$$

5.5.3 Composite Gutter Sections

The design of composite gutter sections requires consideration of flow in the depressed segment of the gutter, Q_w. Equations are provided for use to determine the flow in a width of gutter in a composite cross section, W, less than the total spread, T.

For a composite street section as given in Figure 5.5:

$$Q = Q_w + Q_x \tag{5.17}$$

Q_w = Flow rate in the depressed section of the gutter (m^3/s) (flow within gutter width, W)
Q_x = Flow capacity of the gutter section above the depressed section and within the street width, T_X, (m^3/s)

$$Q = \frac{Q_x}{(1 - E_0)} \tag{5.18}$$

Where,

$$E_0 = 1/\left\{1 + \frac{S_W/S_X}{\left[1+\frac{S_W/S_X}{\frac{T}{W}-1}\right]^{2.67} - 1}\right\} \tag{5.19}$$

And,

$$S_w = S_x + a/W \tag{5.20}$$

Where,
Q = Gutter flow rate (m^3/s)
E_0 = Ratio of flow in a chosen width (usually the width of a grate) to total gutter flow (Q_w/Q)
W = width of the gutter (typical value = 0.6 m)
S_W = the gutter cross slope (typical value = 1/12)
a = gutter depression = $WS_W - WS_x$
Figure 5.5 depicts all geometric variables. From the geometry, it can be shown that:

$$Y = a + TS_x \tag{5.21}$$

And,

$$A = \frac{S_x T^2 + aW}{2} \tag{5.22}$$

Where,

y = flow depth above depressed gutter section (m). Note that the depth of flow at the gutter line is defined as d, where d = y + a
A = flow area (m^2)

Illustrative Example 5.6

Determine the discharge in a composite gutter section if the allowable spread is 3 m, the gutter width is 0.6 m, and the vertical depth between gutter lip and gutter is 0.05 m. The street's longitudinal slope is 1%, the cross slope is 2%, and the curb height is 0.2 m.

Solution:

First determine the gutter cross slope, Sw, using Equation 7-8:

$$S_w = S_x + \frac{a}{W}$$

$$S_w = 0.02 + \frac{\left(\frac{0.6}{12} - 0.6 \times 0.02\right)}{0.6} = 0.083\ m$$

The flow in the street is found as:

$$Q_x = \frac{K_c}{\eta} S_x^{5/3} S_L^{1/2} Tx^{8/3}$$

$$Q_x = \frac{0.376}{0.012} 0.02^{5/3} 0.01^{1/2} 2.4^{8/3} = 0.047$$

$$E_0 = \frac{1}{\left\{1 + \frac{0.083/0.02}{\left[1 + \frac{0.083/0.02}{\frac{3.6}{0.6} - 1}\right]^{2.67} - 1}\right\}} = 0.4920$$

Now the theoretical flow rate can be found as:

$$Q = \frac{Q_x}{(1 - E_0)} = \frac{0.047}{(1 - 0.492)} = 0.0925$$

Then, the computed flow depth is:

Y = a +TS_x = 0.036+ 3.6×0.02 = 0.108 m

5.6 Storm water inlets

Storm water inlets are devices used to collect runoff and discharge it to an underground storm drainage system. Inlets are suitably located on pavements, in gutter sections, paved medians, road side and at locations of specific requirement.

5.6.1 Types of inlets

i. Kerb inlet

Kerb inlets are vertical openings in the road kerb when they are equipped with the diagonal notches cast into the gutter along the kerb opening to form a series of ridges or deflectors. Such inlets are suitable where heavy traffic is expected.

ii. Gutter inlets

Gutter inlets are horizontal openings covered with one or more suitable gratings through which the flow passes.

iii. Combination inlets

Combined grate and curb inlets are more efficient. These are compound of a curb and gutter inlet acting as a single inlet. Following figures give the details of different types of inlet as shown below:

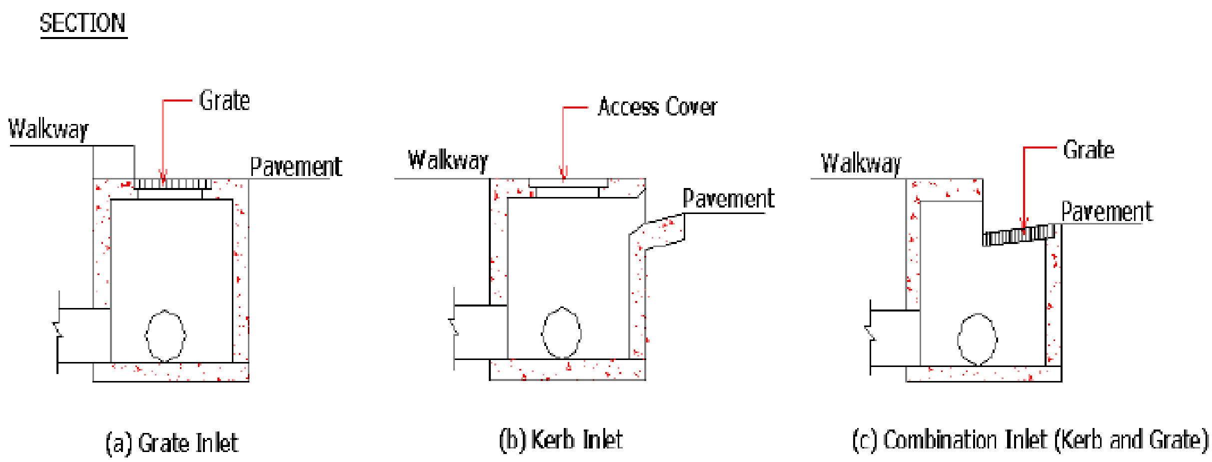


Figure 5.6: Section of Street Inlet

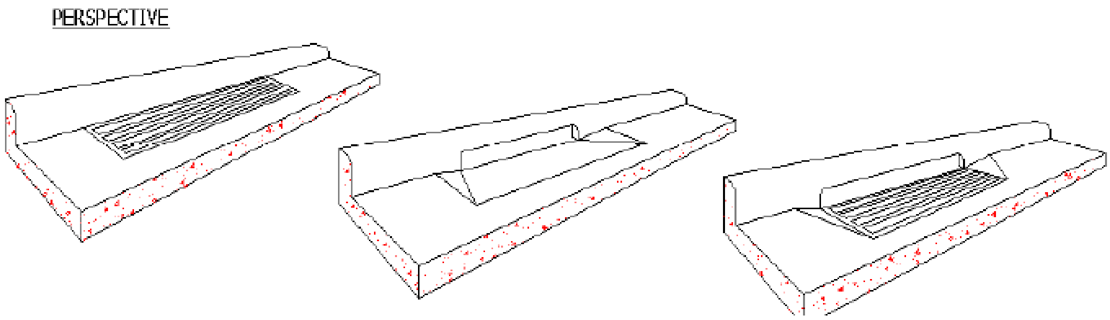


Figure 5.7: Plan of street inlet

iv. **Catch basin**

The catch basin illustrated in Fig 5.8 is a special type of inlet structure designed to retain sediment and debris transported by storm water which might enter into storm water system and clog the storm pipes. A separate catch basin may be used for each street inlet or to save expenses, the pipes from several outlets at a corner may discharge into the same catch basin. Catch basin sumps require periodic cleaning to be effective and if not properly maintained they may become odorous and mosquito nuisance.

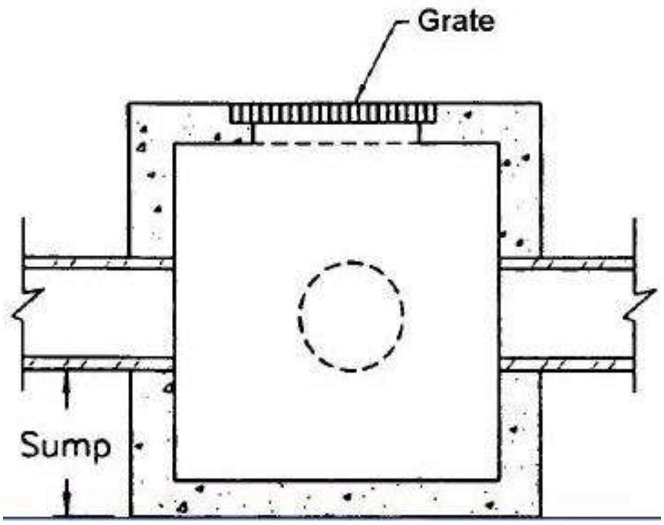


Figure 5.8: Catch Basin

5.6.2 Street Inlet

Street Inlets admit storm runoff to storm water drains. They are designed to remove the flow in gutters with minimum interference to traffic. There are 3 general types of inlets Kerb Inlets, Gutter Inlets and Combination Inlets.

On the basis of model studies, empirical formulae are developed for flow into gutter inlets and kerb inlets with and without depression as follows:

$$\frac{Q}{L} = 0.04176 \times d^{0.5} \tag{5.23}$$

$$\frac{Q}{L} = 1.296 \times i^{0.579} \times \left(\frac{Q_o}{\sqrt{S}/n}\right)^{0.563} \tag{5.24}$$

Where,
Q_o : Discharge into inlet, m^3/sec
Q : Flow in Gutter, m^3/sec
l : Length of the opening, m
i : Cross slope of the gutter
S : Longitudinal slope
d : Depth of flow in Gutter, m

Illustrative Example 5.7:

For a flow of 0.0283 m^3/sec, a longitudinal street grade of 2 %, a mean crosswise street grade of 5.6% and a manning coefficient of 0.015, find (a) length of an undepressed kerb inlet required to capture 90% of a flow, and (b) maximum depth of flow in gutter

Solution:

$$\frac{Q}{L} = 1.296 \times i^{0.579} \times \left(\frac{Q_o}{\sqrt{S}/n}\right)^{0.563}$$

$$\frac{Q}{L} = 1.296 \times 0.056^{0.579} \times \left(\frac{0.0283}{\sqrt{0.02}/0.015}\right)^{0.563} = 0.00928$$

For 90% capturing

$$\frac{0.9 \times 0.0283}{L} = 0.00928$$

Hence, L = 2.74

Calculate depth:

$$\frac{Q}{L} = 0.04176 \times d^{0.5}$$

$$\frac{0.0283}{2.74} = 0.04176 \times d^{0.5}$$

d = 0.06 m

The kerb Inlet is designed for length 2.74 m and depth of flow in the gutter at the kerb Inlet is 0.06 m

5.6.3 Location of inlets

Inlet structures are located at the upstream end and at intermediate points along the gutter line. Inlet spacing is controlled by the geometry of the site, inlet opening capacity and tributary drainage magnitude. Inlet placement is generally a trial and error procedure that attempts to produce the most economical and hydraulically effective system.

Following rules may be observed while locating storm water inlets:

- Inlets are constructed from the upper most point of the gutter section, successively spaced by locating the point where, some of the bypassing flow and the flow from the additional contributing area, exceed the gutter capacity.
- Inlet should be placed at intersections to prevent street cross flow which could cause pedestrian and ventricular traffic hazards.
- Inlets are also required where the street cross slope begins to super elevate.
- Inlet should be located at any point where side drainage enters streets and may overload gutter capacity
- Inlets are required to be constructed at all low points in the gutter grade and at median breaks.
- Inlets should be located upstream of the bridges to prevent storm flow on to the bridge deck and down steam of bridges to intercept drainage from the bridge.
- As a matter of general practice inlets should not be placed within driveway areas.

Illustrative Example 5.8:

Determine inlet spacing to cater runoff from half road catchment. Following data are given:

- Rainfall intensity for Design storm, I = 300 mm/hr
- Half road width = 9 m
- Longitudinal slope = 0.5 %

- Cross slope = 3%
- Width of gutter = 1.5 m \
- Runoff coefficient = 0.91

Solution:

1) Runoff as per Rational formula (Q_{road}) = CIA/360

= 0.91 x 300 x (9 x L_1 x 10^{-4})/360

= 0.000683 L_1

where L_1 is the length of gutter flow in the upstream subcatchment.

(3) Calculate the allowable limit of gutter flow.

Compute the gutter discharge, Q, using equation

$$Q = \frac{K_c}{\eta} S_x^{5/3} S_L^{1/2} T^{8/3}$$

Where:
K_c = empirical constant equal to 0.376
η = Manning's roughness coefficient = 0.013
T = Width of flow (spread) = 1.5 m
S_x = Cross slope = 0.003
S_L = Longitudinal slope = 0.005

Using the Design Chart 24.1 and W = 1.5 m;
Q = 0.018 m3/s
= 18 L/s and V x D is less than 0.4 m/s.
Therefore, spacing for the first inlet is,
L_1 = 0.018 / 0.000683
= 26.3 m » 26 m

Therefore, the inlet spacing to be adapted is 26 m.

5.7 Gradually Varied Non-uniform Flow

If subcritical flow exists in a channel of mild slope and this channel meets with a channel of steep slope in which the normal depth is super critical then there will be change of surface level between the two. In this situation the water surface level changes gradually between the two. The flow in the joining region is known as gradually varied flow. And if the situation is reversed that is upstream slope is steep with a supercritical flow and downstream with a sub critical flow, then there must occur a hydraulic jump to join the two. There may occur a short length of gradually varied flow between the channel junction and the jump. The above situations are shown in the figure 5.9.

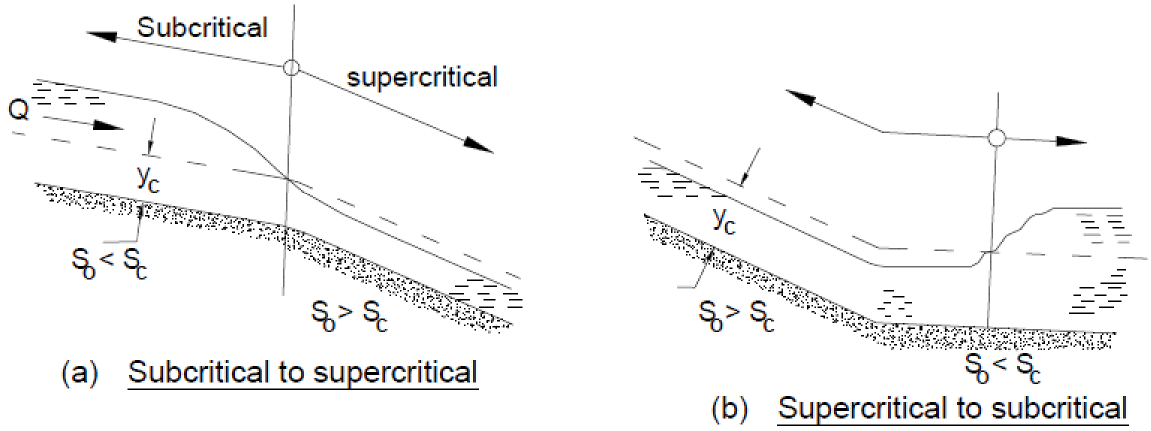


Figure 5.9: Examples of Subcritical & Supercritical Flow

Similarly if a storm channel discharges in a river/ stream, two situations may arise (1) when river/stream surface level is below the invert of discharging channel (2) when surface level of stream/ river is above the invert of the storm channel and above the surface level of water in the channel.

In the first case a draw down curve type of profile develops with gradually varied flow originating from the point of drop backward. In the second case, a back water curve develops and the profile can be determined from analysis. Channels/ conduits should be designed considering water profiles under these conditions.

5.7.1 Basic equation for gradually varied flow

The basic assumption in the derivation of gradually varied flow is that the change in energy with distance is equal to the frictional losses. Based on the above assumption the gradually varied flow equation can be given as follows:

$$\frac{dY}{dX} = \frac{S_o - S_f}{1 - F_r^2} \qquad (5.25)$$

Where,
F_r is the Froude Number
Y is depth of flow
X is the distance along flow alignment
S_o is the bed slope
S_f is the friction losses

The basic equation of gradually varied flow describes variation of depth Y with distance X in terms of the bed slope. So, also the frictional lossS_f, the discharge Q and channel shape.

The differential equation of gradually varied flow as derived above has no explicit solution except numerical integration method which is the only practical solution.

5.7.2 Method of solution of the gradually varied flow equation

There are two basic numerical methods that are used to solve the above flow equations:

1. Direct Step Method – Distance from depth.
2. Standard Step Method – Depth from distance.

5.7.2.1 Direct Step Method

The Direct Step Method is recommended for prismatic channel/pipe conditions since it does not involve tedious iterative operations. In the Direct Step method an increment of water depth is chosen, and the distance over which the depth change occurs is computed. It is appropriate for prismatic channel sections which occur in most conduits, and can be useful when estimating both supercritical and subcritical profiles. For supercritical flow, the water surface profile is computed downstream. For subcritical flow, the water surface profile is computed upstream.

The equation used is

$$\Delta \mathrm{E_s} = \Delta \mathrm{X}(S_o - S_f) \qquad (5.26)$$

Following steps may be followed:

- Assume value of control depth
- Calculate the specific energy E_{sg}
- Calculate S_f (mean)
- Calculate ΔE_s and calculate corresponding ΔX which is the distance from the starting point and subsequently between intervals

Illustrative Example 5.9:

A 3.0 m diameter circular out fall storm conduit laid on a gradient of 1 in 2000 discharges 5.0m^3/sec in a tidal river. During tidal hours the water level of the river rises 2.0 m above the invert of the incoming conduit whereas during non-tidal hours the water level of the river recedes below 1.5 m below the invert of the incoming conduit. Determine water surface profile generated inside the conduit during tidal and non-tidal hours. Assume n = 0.013

<u>Solution (i) for Back water curve :</u>

Discharge = 5.0m^3/sec
Diameter = 3.0m
Slope = 0.0005
By applying formula Q=A$^{5/3*}$s$^{1/2}$/n *P$^{2/3}$
At full flow discharge= 10.0 m^3/sec approximately

Calculate depth of flow at 5 m^3/sec discharge.
As, $\frac{q}{Q} = 0.5$ So, from Table 5.5, $\frac{d}{D} = 0.5$
d = 0.5 × 3 = 1.5 m

During tidal hours the river water rises 2.0 m above the invert of the conduit which will create a back water curve of surface level of water inside the conduit. To plot the profile of back water curve, computations are given in the following table along with corresponding plot of back water profile under gradually varied flow condition. Obtain depth, area, hydraulic radius from given table of geometric element for circular channel section.

Table 5.9: Calculation of back water curve

S.No.	Bed Slope (So)	Mannings Coefficient (n)	Discharge	Depth	Flow Area (A)	Hydraulic mean radius (R)	Velocity	Specific Energy (Es)	Frictional slope (Sf)(mean)	So - Sf	ΔEs	Cumulative Distance (X)
			m^3/s	m	m^2	m	m/s	(m)	(m)	(m)	(m)	(m)
	1	2	3	4	5	6	7	8	9	10	11	13
1	0.0005	0.013	5	2	4.949	0.8697	1.0103	2.0520222	0.000208	0.000292	0	0
2	0.0005	0.013	5	1.9	4.691	0.8517	1.0659	1.9579091	0.000223	0.000277	0.0941130	339.53
3	0.0005	0.013	5	1.8	4.428	0.8328	1.1292	1.8649869	0.000249	0.000251	0.0929223	709.61
4	0.0005	0.013	5	1.7	4.073	0.8028	1.2275	1.7767939	0.000295	0.000205	0.0881930	1140.02
5	0.0005	0.013	5	1.6	3.804	0.7773	1.3143	1.6880423	0.000352	0.000148	0.0887516	1738.80
6	0.0005	0.013	5	1.5	3.534	0.7500	1.4147	1.6020080	0.000424	0.000076	0.0860343	2871.96

Calculation shall be done as follows:

- Column 1 records Bed slope (S_o) of the conduit
- Column 2 records Manning's coefficient (n)
- Column 3 records discharge
- Column 4 records the control depth which is the water level of river above the invert level of the conduit, 2.0 m. Step Δy is taken as 0.1.
- Column 5 records flow area which can be obtained from the Table given in Appendix 5.3: Geometric elements for Circular Channel Sections
  Example: As, y/d_o = 2/3 = 0.67 for control depth 2 m
  From Table given in Appendix 5.3: A/ d_o^2 = 0.5594
  So, A = 0.5594 × 3^2 = 5.0346 m^2
- Column 6 records Hydraulic mean radius which can be obtained from the Table given in Appendix 5.3: Geometric elements for Circular Channel Sections

Example: As, $y/d_o = 2/3 = 0.67$ for control depth 2 m
From Table given in Appendix 5.3: $R/d_o = 0.2917$
So, R = 0.2917 × 3 = 0.8751 m

- Column 7 records Velocity = Discharge / Flow area
- Column 8 records Specific energy = $y + v^2/2g$
- Column 9 records Sf which is calculated from Manning's formula. $Sf = (v \times n/R^{2/3})^2$
- Column 10 records So - Sf (mean)
- Column 11 records change in specific energy with respect to change in depth of water
- Column 12 records cumulative distance calculated from the formula $\Delta E_s = \Delta X(S_o - S_f)$

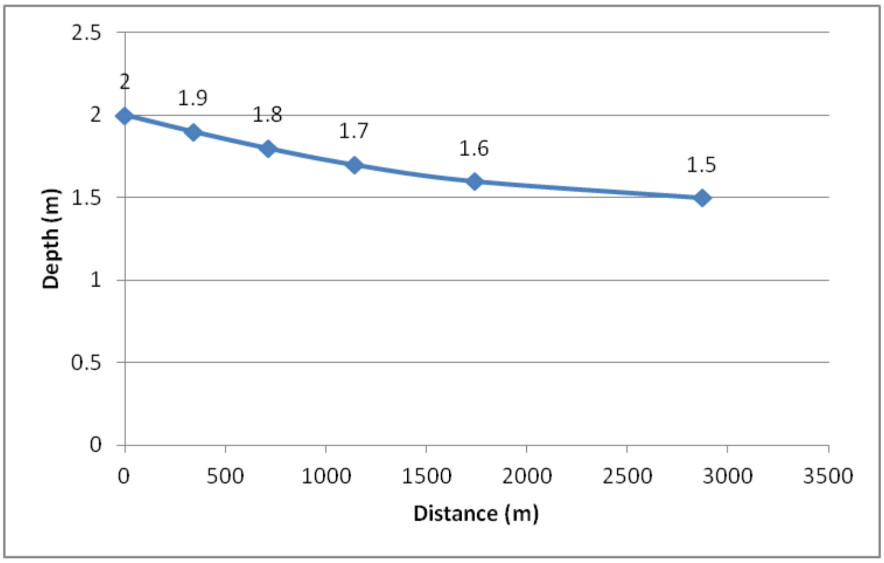


Figure 5.10: Depth Distance Relationship (Back Water Curve)

<u>Solution (ii) for Draw down curve:</u>

When river water during non-tidal hours recede 1.5 m below invert level of incoming conduit, the storm water discharging in the river falls freely, consequently creating a draw down curve starting from the out fall point in the conduit backwards till it attains normal depth. Computations for draw down curve along with graph of draw down curve profile based on the theory and analysis of gradually varied flow conditions in open channel as given in the manual.

For finding the critical depth and other geometric elements from the table given in Appendix A 5.3 containing the geometric elements of circular channel section.

Z (section factor) = $Q/g^{1/2} = 5/9.8^{1/2} = 1.596$

$Z/d_o^{2.5} = 1.596/15.58 = 0.10$, for this $y/d_0 = 0.318$

Hence y (critical depth) = 0.95 m, this is the control depth for computation as given below.

Table 5.10: Calculation of draw down curve

S.No.	Bed Slope (So)	Mannings Coefficient (n)	Discharge	Depth	Flow Area (A)	Hydraulic mean radius (R)	Velocity	Specific Energy (Es)	Frictional slope (Sf)(mean)	So - Sf	ΔEs	Cumulative Distance (X)
			m^3/s	m	m^2	m	m/s	(m)	(m)	(m)	(m)	(m)
	1	2	3	4	5	6	7	8	9	10	11	13
1	0.0005	0.013	5	0.95	1.867	0.527	2.6787	1.3157114	0.0028523	-0.002352	0	0
2	0.0005	0.013	5	1.1	2.291	0.593	2.1821	1.3426829	0.0022330	-0.001733	-0.0269715	15.56
3	0.0005	0.013	5	1.2	2.641	0.643	1.8935	1.3827409	0.0016629	-0.001163	-0.0400580	50.01
4	0.0005	0.013	5	1.3	2.906	0.677	1.7205	1.4508759	0.0012521	-0.000752	-0.0681350	140.60
5	0.0005	0.013	5	1.4	3.174	0.710	1.5752	1.5264576	0.0009572	-0.000457	-0.0755817	305.92
6	0.0005	0.013	5	1.5	3.534	0.750	1.4147	1.6020080	0.0007268	-0.000227	-0.0755504	639.07

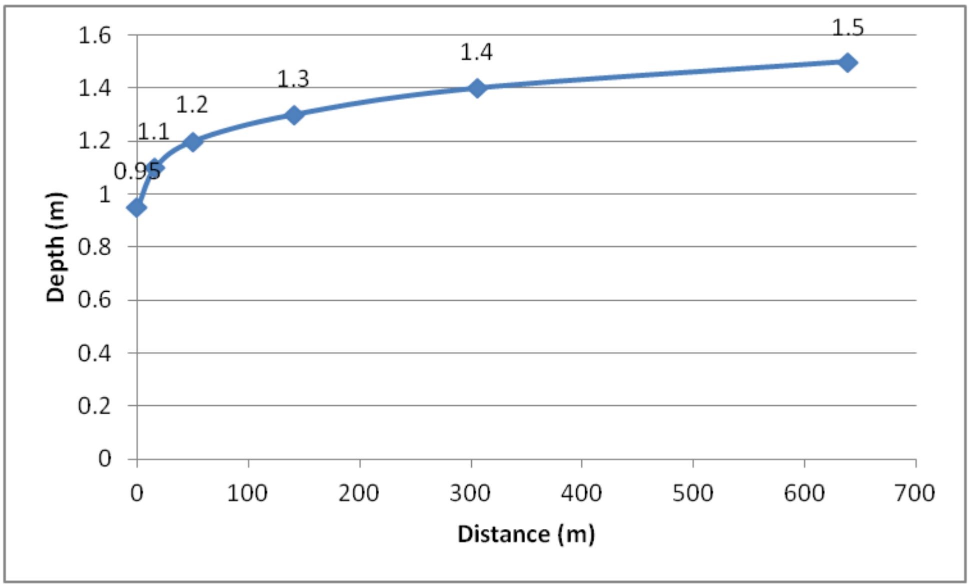


Figure 5.11: Depth Distance Relationship (Draw Down Curve)

5.7.2.2 Standard Step Method

The Standard Step Method on the other hand, is tedious iterative and is usually applicable to non-prismatic channel like rivers and streams.

5.8 Rapidly Varied Non – Uniform Flow

Rapidly varied non uniform flow produces abrupt changes in depth and velocity over very short distances, as in the case of flow over spillway, over sharp crested weir and flow through regions of changing cross-sections. Rapid change can also occur when there is a change from supercritical to subcritical flow in a channel reach at a hydraulic jump.

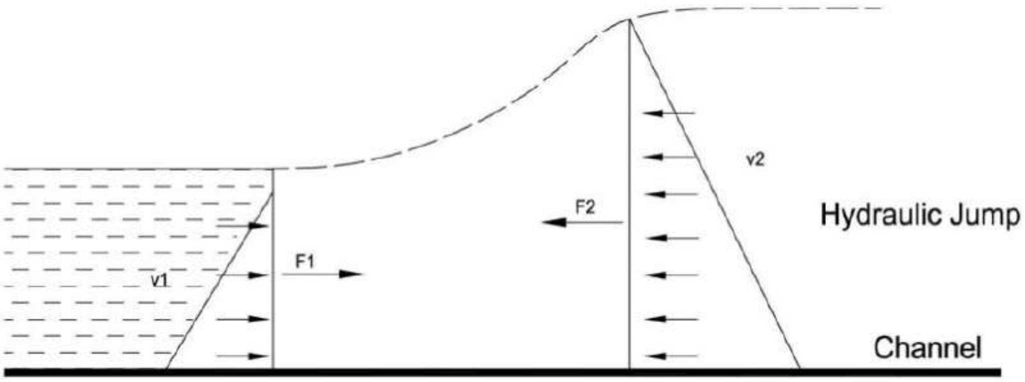


Figure 5.12: Hydraulic Jump

Following equation can be used to compute depth of hydraulic jump when incoming flow depth, velocity and channel geometry are given:

$$y_2 = \frac{y_1}{2}\left[\sqrt{8F_r^{\,2} + 1} - 1\right] \qquad (5.27)$$

Froude number can be determined by the formula given under specific energy section.

5.8.1 Energy Dissipation

Storm pipe drains constructed in plains are commonly designed for sub critical flow developing a self-cleansing velocity that may not cause erosion in channel/pipe or damage hydraulic structures. But in hilly region high velocity of moving storm water down the steep slope causing supercritical flow conditions cannot be avoided and that may result in rapid erosion of channel and damage to the downstream structures. To safeguard against such risks two control measures are generally adopted either to reduce the velocity of flow or dissipate the energy by means of hydraulic jump as described.

5.8.2.1 Stepped Channel

Stepped channels are commonly employed to permit flow along the slopes. They can effectively dissipate the energy and reduce the velocity within safe limits. Design guidelines for such channels may be referred to Appendix A 5.2.

5.8.2.2 Chute

Chutes are constructed to dissipate the energy down the slope where it flattens to gentle slope, resulting in hydraulic jump. This reduces the velocity of flow on the paved apron to a point where the flow becomes incapable of scouring the downstream channel bed. The jump is confined to the channel reach that is known as the stilling basin. In practice the stilling basin is seldom designed to confine the entire length of free hydraulic jump on the paved apron because such a basin would be too expensive. Consequently accessories such as sill etc to control the jump are usually installed in the basin. The main function of such control is to shorten the range within which the jump will take place. The control improves the dissipation of energy function of the basin and stabilises the jump action. Design guide lines may be referred to any standard book on irrigation and hydraulic structures Like Irrigation Engineering and Hydraulic Structures by Santosh Kumar Jain.

5.8.2.3 Aprons

Aprons are provided upstream and downstream of weir in order to protect the scour from reaching to the concrete floor upstream and downstream of the weir.

5.8.2.4 Afflux

The rise in the maximum flood level (HFL) upstream of the weir caused due to the construction of weir is called Afflux. This may occur in storm channels if control structure or obstruction is placed across the channel that extends as backward curve discussed in Gradually Varied Flow phenomenon.

5.8.2.5 Loss of Energy in Hydraulic Jump

Loss of energy in hydraulic jump is calculated from the following formula:

$$Es \text{ (Energy Loss)} = (y_2 - y_1)^3 / 4y_1y_2 \qquad (5.28)$$

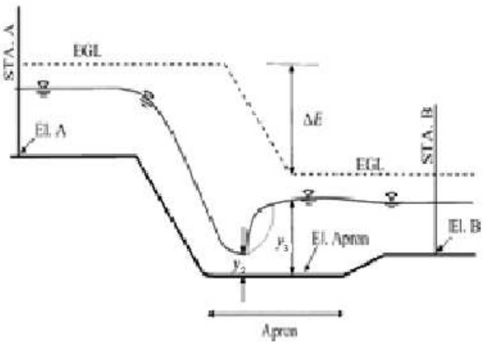
y_1 = Depth of flow before jump
y_2 = Depth of flow after jump

Illustrative Example 5.11

A concrete chute with a stream width of 0.6 m is discharging water down the embankment of 3.0 m height with a steep slope. The discharge is 0.1m^3/s. Find the velocity and depth of water down the slope of the toe level where hydraulic jump takes place. Find also the energy dissipated due to jump.

Since the water is to move down the steep slope, critical depth at critical velocity will be developed at the edge of the fall:

Solution:



Q = $0.1 m^3/s$
B = 0.6m.
A_C = $B \times Y_C$ = $0.6\, Y_C$
Y_C = Critical depth
V_c = Critical velocity.

As per the following formula, at critical depth

$$\frac{Q^2 B}{g A_c^3} = 1$$

Solving the equation,

Y_C is obtained as 0.14 m and V_C = 1.190 m /s

$$E_s = Y_c + \frac{V_c^2}{2g} = 0.14 + \frac{(1.190)^2}{2 \times 9.81} = 0.212m$$

After 3.0 m drop, energy level at the toe level is equal to 3 + 0.212 = 3.212 m

$Y_2 + \frac{V_2^2}{2g} = 3.212$ m

$\frac{Q}{0.6}$ = $V_2 \times Y_2$

$\frac{0.1}{0.6}$ = $0.167 = V_2 \times Y_2$

$V_2 = \frac{0.167}{Y_2}$

$$3.212 = y_2 + \frac{\left(\frac{0.167}{y}\right)^2}{2g}$$

$Y_2 = 0.021$

$V_2 = 7.9$ m/s

Froude Number is calculated by the formula:

$$F_r = \frac{v}{\sqrt{gy}} = \frac{7.9}{0.453} = 17$$

$Y_2 = 0.021$

$$y_3 = \frac{Y_2}{2}\left\{\sqrt{8Fr^2 + 1} - 1\right\}$$

$$= \frac{0.021}{2} \times \left\{\sqrt{8 \times 17^2 + 1} - 1\right\}$$

$= \frac{0.021}{2} \times 47.09 = 0.49\text{m}$

$V_3 \quad = \frac{0.1}{0.6 \times 0.49} \quad = 0.34\text{m/s}.$

Loss of energy = $(y_3 - y_2)^3 \;/\; 4\, y_2\, y_3$

$= 2.51\text{m}$

5.9 Hydraulic routing

The flow of storm water through channel and conduit occurs in state of unsteady condition and its flow rate, velocity and depth vary in space and time throughout the channel/ conduit system. To obtain the values of these parameters hydraulic flow routing based on partial differential equations known as saint venant equations for one dimensional flow can be applied. The following contains the summary of these equations neglecting lateral inflow.

Continuity and Momentum Equations

Continuity equation

Conservation form

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0$$

Nonconservation form

$$V\frac{\partial y}{\partial x} + y\frac{\partial V}{\partial x} + \frac{\partial y}{\partial t} = 0$$

Momentum equation

Conservation form

$$\frac{1}{A}\frac{\partial Q}{\partial t} + \frac{1}{A}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + g\frac{\partial y}{\partial x} - g(S_o - S_f) = 0$$

Local acceleration term	Convective acceleration term	Pressure force term	Gravity force term	Friction force term

Nonconservation form (unit width element)

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial y}{\partial x} - g(S_o - S_f) = 0$$

Kinematic wave
Diffusion wave
Dynamic wave

As shown above, alternative hydraulic flow routing equations are formulated by using full continuity equation while eliminating some terms of momentum equation. The simplest hydraulic routing equation is the kinematic wave that includes the gravity and frictional forces only, balancing each other while neglecting other terms. The diffusion wave routing equation incorporates the pressure term only. The dynamic wave equation considers all the terms of momentum equation.

Hydrograph generated by rainfall- runoff simulation as described in chapter 4 is routed by these equations as per requirement of the flow condition through channels/ conduits to the out fall point. The channel/conduit can be sized from resulting computations of flow rate, velocity and depth along any section of the channel/ conduit. The resulting hydrograph can also be generated at the out fall end.

Storm Water Management Model 'SWMM' developed by US EPA is computer software program that is widely used to successfully perform the related computations.

Hydraulic modeling has become increasingly acceptable solutions in planning and design of urban storm drainage systems since these solutions are more sustainable help in designing the system without complicated and time consuming manual computations compared to traditional practices.

5.10 Engineered Channels

Storm drainage system should be designed in an environmentally responsible manner to minimize disruption of the natural environment of the city natural streams / waterways. Engineered waterways / channels are preferred means of meeting the objective by providing a drainage system that more closely resembles natural streams / waterways. These channels are components of the major drainage system designed to collect and convey flows from minor drainage system. The following sections discuss the design guidelines for engineered channels as well as modified natural channels.

Figure 5.13: Concrete Channel

Figure 5.14: Natural Channel

Design guidelines for engineered channels are given in Table 5.11.

Table 5.11: Design Guidelines for Engineered Channels

S. No.	Item	Value
1.	Design Flows	25 years Return Period (as per the past historical data)
2.	Flow Regime	Froude number < 0.8
3.	Design velocity with grassed	1.2 m/sec maximum

	lining- With other lining such as- • Riprap • Concrete	 1.8 m/sec 3.0 m/sec
4.	Maximum Longitudinal Slope	0.4 % for natural lining
		0.2 % for concrete lining
5.	For steep slope	Drop structure may be provided
6.	For curvature into the channel	Centre line of curvature should have minimum radius, 2× top width, but not less than 30 m

Note: Natural channels should be preserved as far as possible and engineering of channels should be minimized.

5.10.1 Curvature and Super Elevation

Super elevation should be provided on outside bend of channel as per the following equation:

$$\Delta y = \frac{V^2 T}{2 g r_c} \qquad (5.29)$$

Where,

Δy : Difference in water surface elevation inside vs outside of curve.
v : Mean velocity
T : Top width of channel section
g : Acceleration due to gravity
r_c : Radius of curvature

5.10.2 Freeboard

Freeboard of engineered channels should be provided as given in table 5.12:

Table 5.12: Freeboard in Engineered Channels

S. No.	**Discharge (m^3/s)**	**Freeboard (mm)**
1	Below 3	450
2	3 and above but below 30	600
3	30 and above but below 300	900
4	300 and above but below 3000	1200
5	3000 and above	1500

Source: IS 7784(Part I): 1993

5.10.3 Modification of Natural Channels

Natural channels are either having steep erodible banks or bottom or mild sloped channels that are almost stabilised. Therefore, if such natural channels are to carry the storm run-off from urbanized areas, some form of modifications of the channel is essentially required to stabilize it. Therefore, following criteria should be ensured while engineering the natural channels:

- Lining of bank and bed if necessary
- Channels and overbank capacity shall be adequate for design storm i.e. 25 year return period.
- Channel velocity shall not exceed 2 m/sec or the critical velocity for any particular section with minimum value of Manning's Roughness coefficient "n", in case of stabilized earthen channel.
- Water surface limits shall be defined so that the flood plain can be zoned and protected.
- Drop structures or check dams should be constructed to limit flow velocities and control water surface profile, particularly for the initial storm run-off.

5.10.3.1 Channel Linings

Channel linings should be provided wherever the bed and banks are not in stabilized condition and likely to be eroded in high floods in natural channel. Different types of channel linings are as follows:

a) Rigid Lining: under rigid lining criteria following type of linings are considered:
   - ✓ Concrete
   - ✓ Precast concrete slab
   - ✓ Stone masonry
   - ✓ Cellular reinforced concrete paving with infill soil.

b) Flexible lining: under flexible lining criteria following type of linings are considered:
   - ✓ Rip-rap
   - ✓ Gravels
   - ✓ Gabion or random Rubble

Each type of lining should be scrutinized for its applicability, how it meets other community needs, its long term integrity, maintenance needs etc. As lining is costly component of a lined channel. Therefore, such shape of channel should be adopted which has less surface area and more hydraulic capacity. Though, semi-circular section provides maximum hydraulic capacity with minimum surface area per unit length, but cost and ease of construction permit preference of trapezoidal section, which is somewhat pragmatic approximation of semi-circular shape, hence adopted for storm water drains/ channels.

CHAPTER – 6: DESIGN CONSIDERATIONS FOR SPECIAL AREAS

6.1 General

Storm runoff estimation for hydraulic design of storm water drains is elaborated in Chapter 4. However, in the course of storm water drains design there are many special areas like hilly and coastal terrains which require special consideration in the design. This chapter outlines the additional design criteria for storm water drains under some specific conditions such as control of erosion & sedimentation, dissipation of excess energy of runoff, design of outfall in coastal areas and control of backflow to minimize incidences of water logging.

6.2 Hilly Terrain

Hilly areas are characterized with high terrain slope. In case of storm, the runoff gushes down the hill at very high velocities causing erosion of soil along drains / slopes. The amount and size of soil particles transported, increase the volume and velocity of runoff and are subsequently carried along drainage system of the basin to the receiving bodies eg, river and stream.

6.2.1 Impact of Erosion and Sedimentation on performance of storm water drains

On steeper slopes, water moves faster as compared to flatter slopes and this increased flow velocity aided by a lack of significant vegetative cover results in transportation of larger amount of sediments. Increasing urbanization of hilly areas increases the paved surfaces and results in increased surface runoff, further aiding the transport of eroded sediments. Deposition of such eroded sediments inflicts serious problems in the drainage channels in the areas located downstream of it resulting in frequent flooding. Rapid downward movement of sediment-laden water can cause problems like landslides that frequently occur during monsoon almost in most of the hilly towns, causing loss of lives and damage to property. Therefore, the interlinked consequences of urban development are transforming the hilly urban watersheds into multi-hazard zones. Protecting erosion and prevention of sedimentation is extremely important in planning and design of urbanization in hilly areas. The following section mentions about the considerations to be adopted in the design of storm water drains to mitigate the issue of erosion and sedimentation.

6.2.2 Additional Design considerations for drainage systems

i. Storm drains should be constructed on both sides of the road and connected with cross drains across the road at suitable intervals having gratings to collect rain water from surface of the road during rains.

ii. Roof water drains should be connected to these drains so that the rain water may not spill over the slopes.

iii. The collected storm water conveyed through drains should be disposed off in the valley stream through existing natural channels or constructed channels along the slope at suitable location.

iv. Natural channels should be engineered either by constructing stepped channel or chute (design may be seen in chapter 5). The width of such engineered channel should never be reduced from its existing natural width.

v. Valley stream bank at the point of outfall should be protected by revetment against erosion.

vi. To drain out the increased discharge through the natural channel in valley, the bank of such channel should be protected by retaining wall made of rock block or gabion box, depending on the steepness of the side slope.

vii. Sufficient weep holes should be provided in case of concrete/masonry retaining walls. Weep holes shall be provided in cement stone masonry walls at spacing of about 1.5 m centre-to-centre in either direction. The size of weep holes shall be 100 mm to 150 mm connected with PVC (flexible) pipes embedded at 10^0 down from the horizontal towards valley.

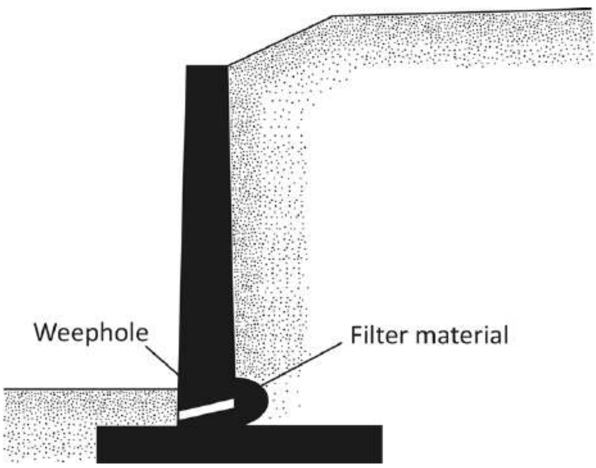


Figure 6.1: Weep holes in a retaining wall

viii. For a channel carrying debris and having moderate slope (say 10° < S < 30°) intermediate sill projecting from the bed can be constructed to reduce the flow velocity.

ix. For a channel having thick natural cover of boulders (which is found in most of the natural channels) no additional measures are necessary for protecting against scouring.

x. In absence of a boulder bed in a moderate slope, discrete concrete block may be placed to prevent scouring under the impact of high stream flow velocity, while keeping the bed permeable to allow infiltration.

xi. Road must have adequate cross slope or camber as per Clause 5 of IRC, SP-42 (1994) for quick disposal of storm water runoff laterally to the road side drain / drains.

xii. A minimum longitudinal grade of 0.5 % should be provided to the road, wherever possible in order to facilitate surface drainage.

xiii. Intercepting drain, as in a hilly terrain sloping towards the road, should have adequate size and be connected properly with well-designed culverts/bridges.

xiv. In case width of terrain contributing flow to the drain is very large, intercepting drain at higher elevation should be provided.

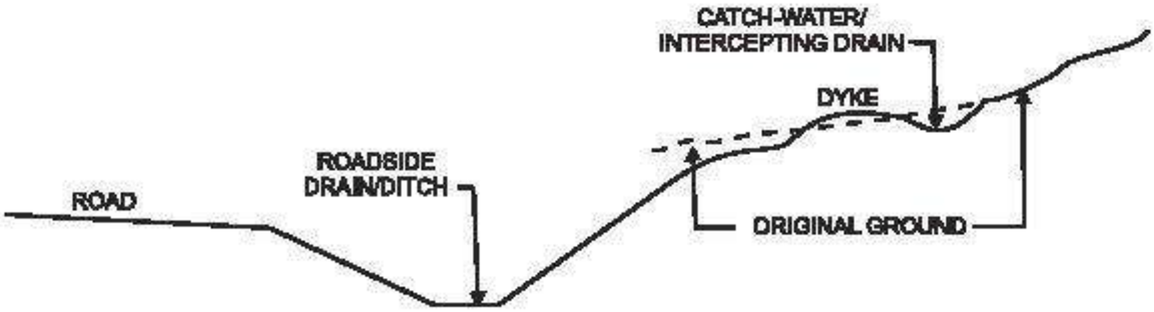


Figure 6.2: Intercepting / Catch Water Drain on a Hill Slope

xv. To dissipate the energy of flowing water with high velocity down the steep slope in hilly area, a stepped channel or chute should be provided with a protective apron as given in clause 5.6.1 of chapter 5.

6.2.3 Temporary erosion and sedimentation control practices

The temporary erosion and sedimentation control measures to be implemented in the catchment area and the drainage systems are elaborated below:

a) Mulching

Mulching refers to the application of plant residues or other suitable materials to the soil surface to prevent erosion and reduce surface flow velocities. Mulching also fosters plant growth by increasing available moisture and providing insulation against extreme heat or cold. Mulch materials, application rates and specifications are

Figure 6.3: Mulching

explained in the Table 6.1.

Table 6.1 Mulch materials, application rates and specifications

Sr. No	Mulch Material	Quality Standards	Application rate	Remarks
1.	Straw	Air-dried, free from undesirable seed and coarse material	50 mm - 75 mm thick, 2000-3000 Kg per 4000 m^2	Cost effective protection when applied with adequate thickness. Hand-application generally requires greater thickness than blown straw. Straw should be crimped to avoid wind blow. The thickness of straw may be reduced by half when used in conjunction with seeding.
2.	Wood Fiber Cellulose	No growth inhibiting factors	450 - 680 Kg per 4000 m^2	Shall be applied with hydro mulcher. Some wood fiber with very long fibers can be effective at lower application rates
3.	Compost	No visible water or dust during handling. Must be purchased from supplier with Solid Waste Handling Permit.	50 mm thick min, approx. 100,000 Kg per 4000 m^2	More effective control can be obtained by increasing thickness to 75 mm.

Source: King County, Washington Surface Water Design Manual

b) Temporary/permanent seeding

Temporary seeding and permanent seeding are two types of vegetative controls. Temporary seeding is applied in areas that will be dormant for 15 days or more whereas, permanent seeding is applied in areas that will be dormant for one year or more. Selection of vegetation types depends on the season, site conditions and costs.

Construction guidelines:

- Prior to seeding, install all necessary erosion control practices such as dikes, waterways, and basins.
- Provide proper shaping of the area to be seeded in a manner such that seedbed preparation and seeding operations can be carried out.

- Soil conditions needed for the establishment and maintenance of seeding must be as follows:
  - Sufficient fine grained material to maintain adequate moisture and nutrient supply.
  - Sufficient pore space (crumb like structure or bulk density 1.2 to 1.5 gm/cm^3) to permit root penetration.
  - Sufficient depth of soil to offer an acceptable root zone. The depth to rock layers shall be 0.3 m or more.
  - A promising pH range for plant growth. If the soil is so acidic then soil modification would be mandatory.
  - Freedom from toxic materials harmful to plant growth.
  - Freedom from excessive amounts of roots, branches, large stones and trash of any kind.

c) Sediment Basins

A sediment basin is a constructed embankment of compacted soil across a drainage way which detains sediment-laden runoff. The basin allows runoff to pond and sediment to settle out. They are generally used where construction area is disturbed in 2 ha or more. Accumulated sediment within the basin should be removed as necessary.

Figure 6.4: Sediment Basins

Design Steps:

When constructing a sediment basin, designers should estimate the site constraints that could affect the efficiency of the sediment basin. These constraints include:

basin capacity, estimated sediment load, and freeboard, maintenance frequency, and hydraulic capacity of outlet structure.
Design sediment basin(s) using the equation:

$$A_s = \frac{1.2\,Q}{V_s} \tag{6.1}$$

Where,

A_s = Minimum surface area for trapping soil particles of a definite dimension,
V_s = Settling velocity of the design particle dimension chosen (Vs = 0.0085 cm/s for a design particle size of 0.01 mm at 68°F)
1.2 = Factor of safety recommended by USEPA
Q = Peak basin influent flow rate (m^3/sec), which shall be calculated by Rational formula given in Chapter 4

This method is dependent on the outlet structure design. If the designer chooses to utilize the outlet structure to control the flow duration in the basin, the basin length (distance between the inlet and the outlet) should not be less than twice the basin width; the depth should not be less than 0.9 m nor greater than 1.5 m for safety reasons and for maximum efficiency.

d) Check Dams

Check dams are small temporary dams, constructed across a drainage ditch to reduce erosive runoff velocities of concentrated flows. Check dams are limited to use on small open channels draining 4 ha (10 ac) or less. Sediments should be removed when it reaches approximately half the height of the dam. Check dams should be spaced in the channel so that the crest of the downstream dam is at the elevation of the toe of the upstream dam.

Figure 6.5: Check Dams

Design Steps:

Check dams should follow to the following requirements:

- Check dams should be constructed before surface runoff is directed to the swale or drainage ditch.
- The maximum runoff contributing area to the dam should be lesser than 10 acre.
- The dam maximum height should be 0.6 m.
- The centre of the dam should be at least 15 cm lower than the outer edges.
- The maximum spacing between the dams should be such that the toe of the upstream dam is at the same elevation as the overflow elevation of the downstream dam.
- The check dam should not be used in a flowing watercourse.
- Stone check dams should be built of a well-graded 5 cm to 7.5 cm stone. 2 cm stone on the up gradient side is suggested for better filtering.
- If sensibly installed and monitored, timber check dams may be used and should be constructed of 10 cm to 15 cm logs embedded at least 45 cm deep into the soil. However, stone check dams are generally chosen. The stone has the capability to conform to the channel and settle if scour happens.

Note: Detailed design may be referred from the 'Manual on Artificial Recharge of Ground Water' published by Central Ground Water Board (CGWB), September 2007.

e) Silt Fences

A silt fence is the most widely used temporary sediment barrier. The fence consists of a filter fabric supported by wooden posts or wire mesh. It is placed across or at the toe of a slope to intercept and detain sediment and reduce flow velocities. The maximum effective life of a silt fence is approximately six months. Proper maintenance of a silt fence requires removal of sediment deposits when necessary. Silt fences which decompose or become ineffective prior to the end of the expected useable life should be replaced immediately.

Figure 6.6: Silt Fence

Design Criteria:

- Ensure silt fence height is a minimum of 400 mm above ground level.
- Place supporting posts for silt fences no more than 2 m apart unless additional support is provided by tensioned wire (2.5 mm HT) along the top of the silt fence.
- Ensure supporting posts are embedded a minimum of 400 mm into the ground.
- Always install silt fences along the contour.
- Join lengths of silt fence by doubling over fabric ends around a wooden post or batten or by stapling the fabric ends to a batten and butting the two battens together
- Install silt fence wings at either end of the silt fence projecting upslope to a sufficient height to prevent outflanking.
- Do not use silt fences in catchments of more than 0.25 ha

f) Brush Barrier

A brush barrier is a temporary sediment barrier composed of materials (such as, weeds, vines, root mats, soil, rock, etc.) pushed together at the perimeter of a given site and at the toe of fills. Maintenance measures include inspection following each rainfall and removal of sediment deposits when they reach half of the barrier height.

The height of a brush barrier shall be a minimum of 1 m. The width of a brush barrier shall be a minimum of 1.5 m at its base (the sizes of brush barriers may vary considerably based upon the amount of material available and the judgement of the design engineer). Material larger than 15 cm in diameter should not be used as the non-homogeneity of the mixture can lead to voids where sediment-laden flows can easily pass.

The drainage area for brush barriers should not be greater than 0.1 ha per 30 m of brush barrier length. Additionally, the drainage slope leading down to a brush barrier must be no greater than 3:1 and no longer than 45 m.

Figure 6.7: Brush Barrier

g) Diversion Dike

A diversion dike is constructed of compacted soil and is used to divert runoff to an acceptable location. They are placed either at the top of a disturbed area to divert site runoff, or at the bottom to deflect sediment-laden runoff to a sediment trapping structure. Dikes should be inspected weekly and after rainfall events and repairs made as necessary.

Following are the installation criteria of diversion dike:

- Clear and grub area for diversion dike construction.
- Excavate channel and place soil on down gradient side.
- Shape and machine compact excavated soil to form ridge.
- Place erosion protection (riprap, mulch) at outlet.
- Stabilize channel and ridge as required with mulch, gravel, or vegetative cover.

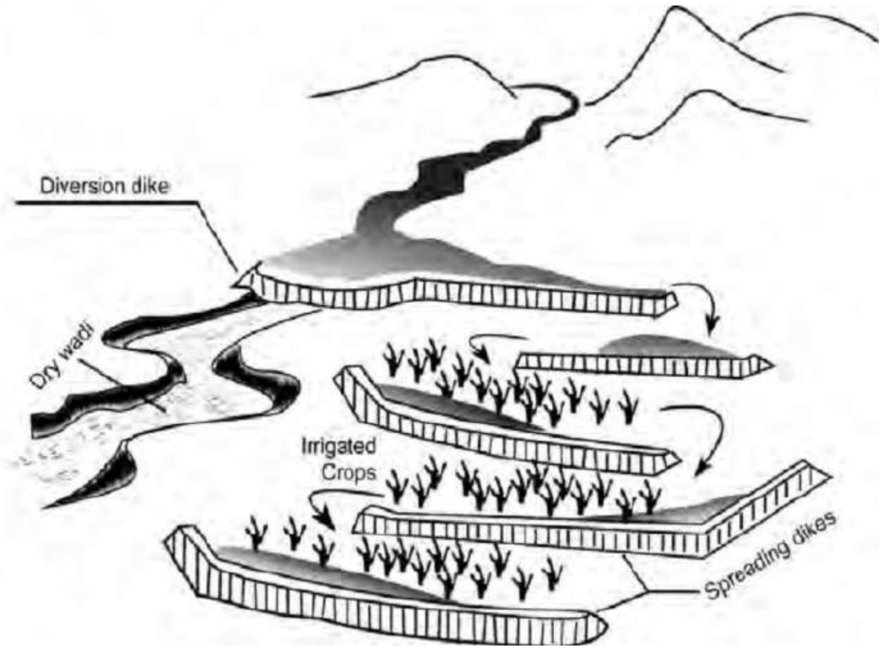


Figure 6.8: Diversion Dike

h) Temporary slope drains

A slope drain is a flexible tubing or conduit used to convey concentrated runoff from the top to the bottom of a disturbed area without causing erosion on or below the slope. It can also be used to carry storm water down a slope away from a control facility. Slope drains should be inspected weekly and after rainfall events to ensure proper operation.

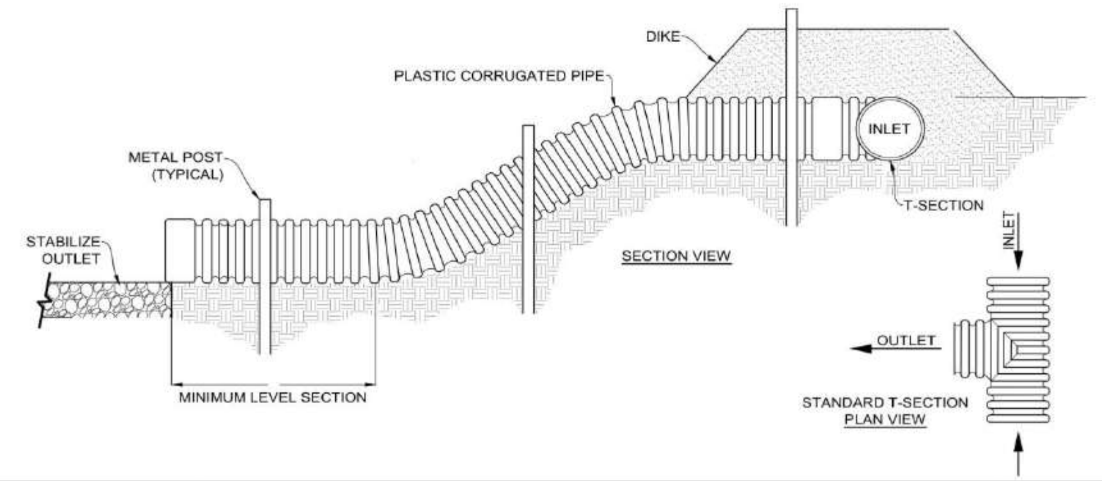


Figure 6.9: Temporary Slope Drain

The temporary slope drain must be sized to safely convey the desired flow volume. At a minimum, it should be sized to convey the 2 year, 24 hour storm.

Temporary slope drains may be constructed of flexible or rigid pipe, riprap, or heavy plastic lining. When piping is used, it must be properly anchored by burying it with adequate cover or by using an anchor system to secure it to the ground.

The discharge from the slope drain must be directed to a stabilized outlet, temporary or permanent channel, and/or sedimentation basin.

6.3 Coastal Terrain

The coastal areas present a unique challenge to the design of storm water drainage systems, owing to tides, high groundwater tables and relatively flat terrain. During high tides, low lying areas along the coast are prone to flooding / inundation and disposal of storm water becomes a problem. During severe storms / cyclones, the discharge of storm water through drains is often not effective and results in water logging and flooding. Backflow of sea water further exasperates the problem. The following section mentions the special design considerations in storm drainage outfalls in coastal areas.

6.3.1 Design Considerations for storm water drainage in Coastal Areas

The following measures suggested should also be incorporated in design for efficient discharge of storm water in coastal areas.

- Encouraging infiltration through low impact development activities, such as, preserving & recreating natural landscape features, bio-retention facilities, vegetated rooftops, permeable pavements, etc.
- Installing pumps to provide sufficient pressure to storm water to overcome backflow during sea level rise. Alternatively, a suitable pumping system or a combination of systems shall be implemented so as to minimize flooding in low lying areas during high tide periods.
- Installing check valves/ flap gates to only allow outflow from storm conduits and effective prevention of back flow.

- **Trench drains** are recommended in locations where there is localized flooding at a low point in a paved area.
- Avoid pipes discharging on beaches.
- Avoid construction of conduit/channels along the shoreline.
- Minimize the number of outlets into the sea or estuary.

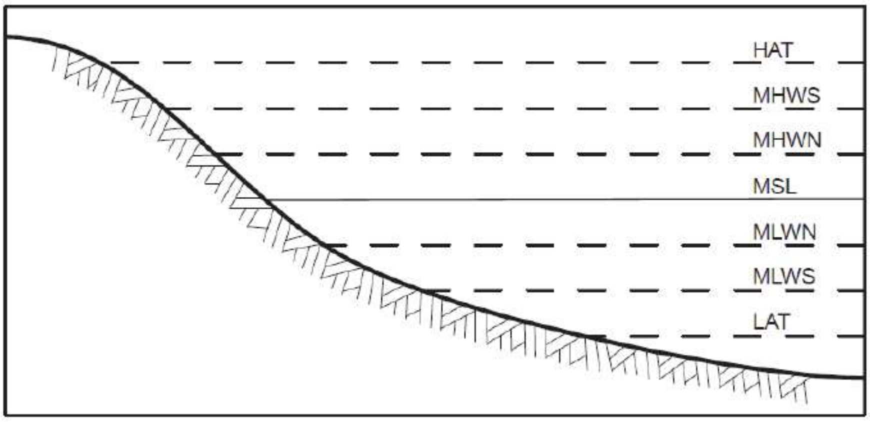
Figure 6.10:Trench Drain

6.3.2 Storm outfalls in coastal towns/cities

Tide levels can influence storm water drainage planning since a significant portion of the drainage infrastructures in the coastal cities are dependent on outfalls having tidal influence of the surrounding sea and estuaries. The establishment of engineering design values for the still water levels or tail water elevations, utilized in drainage outfall hydraulic analysis for storm water management planning should also account for rising sea level trends over a projected period of time to ensure that the planned drainage improvements will function effectively under current tidal water conditions, as well as, future conditions.

6.3.3 Sea levels in different conditions

Variation in sea water levels is essentially required to be understood in its various phases of tidal events before concluding the desired sea water elevation for positioning the storm out fall levels in coastal areas of the sub-continent. The pictorial illustration of the tidal levels is shown in Figure 6.11. The following sections briefly discuss the various relevant aspects.



Source: Adapted from 'Queensland Urban Drainage Manual'

Figure 6.11: Sea levels

a) Tidal water levels

Chart datum is plane below which all depths are published on a navigational chart. It is also the plane to which all tidal heights are referred, so by adding the tidal height to the charted depth, the true depth of water is determined. By international agreement chart datum is defined as a level so low that the tide will not frequently fall below it.

b) Mean sea level

Mean sea level is average level of the sea surface over a long period normally 19 years or the average level which would exist in absence of tide i.e. the average height of surface of the sea at a tide station for all stages of the tide over years of period usually determined from hourly height readings measured from a fixed predetermined reference level (chart datum).

c) Highest Astronomical Tide (HAT) and Lowest Astronomical Tide (LAT)

The highest and lowest tide levels respectively can be predicted to reach under average meteorological conditions or under any combination of astronomical conditions but are not the regular occurrence. These levels will not be reached every year.

d) Spring Tides

During the full moon and new moon phases of a Lunar cycle (approximately a fortnightly occurrence), the gravitational forces of the moon and the sun act to reinforce one another. The tides experience an increased range during these phases. Since the combined tidal force is increased, the high tides are higher and the low tides are lower than the average. Spring tide is a term which implies a welling up of the water and bears no relationship to the season of the year.

e) Neap Tides

The tides of decreased range occurring near the times of first and third quarter phases of the moon when the gravitational forces of the moon and the sun counteract each other. As the combined tidal force is decreased, the high tides are lower and the low tides are higher than average. Neap comes from a Greek word meaning scanty.

f) MHWS (Mean High Water Springs) & MLWS (Mean Low Water Springs)

The height of mean high water springs is the average of the heights of two successive high waters during those periods of 24 hrs (approximately once a fortnight) when the range of the tide is greatest. The height of mean low water springs is the average height obtained by the two successive low waters during the same period i.e.,

MHWS The average height of the high waters of spring tides above Chart Datum.

MLWS The average height of all low waters of spring tides above Chart Datum.

g) MHWN (Mean High Water Neaps) & MLWN (Mean Low Water Neaps)

The height of mean high water neaps is the average, throughout a year as defined above, of the heights of two successive high waters during those periods (approximately once a fortnight) when the range of the tide is least. The height of mean low water neaps is the average height obtained from the two successive low waters during the same periods, i.e.,

MHWN The average height of the high waters of neap tides above Chart Datum.
MLWN The average height of the low waters of neap tides above Chart Datum.

h) Storm surge

A storm surge (meteorological tide) is an atmospherically ocean driven response by extreme surface winds and low surface pressure associated with severe weather conditions, usually cyclones. Strong off-shore winds can generate significant ocean currents, when these currents approach a barrier such as shore lines, sea levels increase as the water is forced up against land. The low atmospheric pressure associated with cyclones can also raise sea levels well above predicted tide levels.

i) Historical sea levels

Historical sea levels are instrumental records of sea level changes measured with permanent tide gauges at required locations. Such gauges are installed at 28 locations along Indian coast line operated and maintained by Survey of India.

j) Availability of Tidal Data

The annual and monthly MSL tide gauge data can be obtained from Permanent Service for Mean sea level (PSMSL) and the satellite altimetry data from Topex/ Poseidon Jason 1 and Jason 2. There are 28 tide gauge stations along Indian coast line. Necessary data can be obtained, recorded by these gauges from Survey of India, Dehradun. Global data can also be obtained from Water Ocean Circulation Experiment (WOCE), National Oceanographic Data Centre (NODC), and Indian National Centre for Ocean Information Service (INCOIS), Hyderabad etc. Besides, national port and harbor authorities of maritime board of coastal states of India also publish annually, tide tables recorded at the minor ports within jurisdiction of their state.

6.3.4 Global trend in sea level variation

Based on major findings of IPCC scientific assessment of working group (2014), it is anticipated that if greenhouse gas emissions continue to increase at the current rate, an average rate of global sea level rise between 3 cm and 10 cm per decade is predicted over the next century, resulting mainly from thermal expansion of the ocean.

6.3.5 Regional trend in sea level variation

Recent studies on regional sea variation along the Indian coast (APAC-2015) reported that average rise in MSL in Bay of Bengal and Arabian Sea based on available tide gauge data-set have been found to be 1.92 mm/year and 1.72 mm/year respectively.

6.3.6 Design considerations for tidal out fall in ocean and bays

Selection of appropriate tail water level at the location of storm water out fall is the basic necessity in design and planning of storm water drainage system of coastal cities. The nature of tidal variation, storm surges, mean sea level, wave set up and climate change effect, that significantly influence tail water level, have been briefly described in the foregoing sections.

However, local maritime board of state Government and/or other local authorities should be consulted to establish an appropriate tail water level for design of storm outfall to ocean/ bay. Consideration should also be given to the joint probability of occurrence of the design storm, tide level and storm surge. The effect of increased tail water level resulting from climate change should be examined and necessary allowance should be made in determining the tail water level. Suggested tail water level for discharge to tidal water ways in design of storm outfall system are given in the Table 6.2.

Table 6.2: Design considerations for tidal out fall

Design condition	Design tail water level
Minor storm (< 1 in 5 years)	In the range of MHWN to MHWS
Major storm (> 1 in 5 years)	In the range of MHWS to HAT
Effect of climate change	Additional 0.3 m

6.3.7 Design consideration for outfall in tidal rivers and streams

In case where the drainage outfall is located in the tidal reach of a stream or river, water levels within receiving waters may be affected by flood flows passing down the receiving waterway. The severity of this coincident flooding will depend principally on the ratio of the time of concentration of the side channel/drain relative to that of the receiving waterway. The procedure described in section 4.4.1.7 (Partial area effect) may be adopted to assess the most critical combination of flows and stream water level. Therefore, the design tail water level should include an appropriate surcharge to the corresponding flood discharge, in addition to the stream water level. Such case may also arise in non – tidal rivers that should be dealt accordingly.

Table 6.3: Design tail water level

Design condition	Design tail water level
Minor storm (< 1 in 5 years)	In the range of MHWN to MHWS
Major storm (> 1 in 5 years)	In the range of MHWS to HAT
Effect of climate change	Additional 0.3 m
Surcharge due to combined discharge of side drain and main stream	As per design calculation in Chapter 4, clause 4.4.1.7 of Partial area effect

Alternatively, local authorities and maritime boards may determine appropriate tail water levels or discharge conditions of particular reaches of tidal streams based on local experience and knowledge.

6.3.8 Design tail water level for non-tidal storm outlets

Design tail water level for the following non tidal water bodies may be adopted as follows:

- For river and streams – High Flood water level (HFL)
- For lakes - Normal high water level or overflow level
- For storm water ponds – Normal high water level or overflow level

6.3.9 Tide gate

Tide gates seal a channel at the outlet to avoid water from flowing backwards through the drainage system. Tide gates are generally provided to prevent against back flow of sea water during high tide, sea level rise and storm surges.

These gates may be circular, square, or rectangular in shape. Usually they are made of cast iron and provided with single or double acting hinges at the top. The project engineer can specify the size, materials, and choice of operating mechanism as required to the manufacturers for procurement and installation.

6.4 River bank protection

River passing through populated areas of towns and cities may cause excessive damage to adjacent land, properties, hydraulic structures etc. due to failure of the banks caused by erosive forces of fast moving currents. It, therefore, necessitates the protection of river banks against erosion and caving resulting in subsequent failure and collapse.

Detailed design and engineering of river bank protection is beyond the scope of this manual. In this regard the provisions of “Handbook for Flood Protection, Anti Erosion and River Training Works” published by Central Water Commission, Government of India are to be followed.

CHAPTER 7: STRUCTURAL DESIGN OF BURIED STORM WATER CONDUITS

7.1 General

Hydraulic design of storm water conduits has been discussed in Chapter 5. These pipes when laid underground are subjected to forces that need consideration of various parameters such as pipe material properties, their supporting strength and various installation and loading conditions like fill loads, superimposed loads, sub-surface water level etc. This Chapter describes the process of structural design of underground rigid and flexible conduits under non-pressure flow application that are generally used in storm water drainage system.

7.2 Type of Buried Pipes

There are two types of conduits that are generally used in storm drainage system namely:

i. Rigid Pipes
ii. Flexible Pipes

7.2.1 Design of buried rigid pipe

Manual on sewerage and sewage treatment published by CPHEEO, 2013 may be referred for design procedure for underground buried rigid pipes.

7.2.2 Design method of buried flexible pipe

Flexible pipe (non-pressure flow) derives its load carrying capacity from its flexibility. Under vertical load, the pipe tends to deflect pressure on soil support along its sides. At the same time, ring deflection relieves the pipe of the major portion of the vertical load, which is then carried by surrounding soil through mechanism of soil arching action over the pipe. Therefore, the design of flexible pipes involves calculation of deflection, buckling and wall thrust under total load including soil load, vehicular load and hydrostatic forces so that the pipe must be able to withstand these forces in order to remain structurally stable.

7.2.2.1 Deflection in flexible pipe

The effective strength of the flexible pipe soil system is remarkably high which is determined by vertical deflection under pipe soil system. As per IS code 16098:2013 (part II), the deflection limit for various classification of PE pipe is given in Table 7.1.

Table 7.1 Recommended Design Deflection Limits

S.No.	Stiffness Class	Average Initial Deflection Percent	Average Long term Deflection Percent
i)	SN 2	5	8
ii)	SN 4, 8,16	8	10

Source: IS code 16098:2013 (part II)

Following formula is applied to compute the vertical deflection in buried flexible pipe for short term and long term conditions:

$$\frac{\Delta y}{D_M} = \frac{K(D_L W_c + W_L)}{\left(\frac{8EI}{D^3}\right) + (0.061 \text{ x } E')} \times 100 \qquad (7.1)$$

Where,

$\frac{\Delta y}{D_M}$: Deflection in %

K : Bedding Constant (dimensionless);

D_L : Deflection lag factor (dimensionless);

W_c : Soil Column load on pipe, kPa

W_L : Live load, kPa

$\frac{EI}{D^3}$: Ring Stiffness in kPa which is designated as SN by IS code 16098:2013 (Part II)

E' : Modulus of soil reaction in kPa

D_M : Mean Diameter, m

(a) Loading on pipe

The loading on buried pipes is composed of dead load and live load i.e

i. Dead load, kPa (W_C)

Dead load is the soil Column load on pipe which is calculated by the following formula:

$$W_c = H \times \gamma_S \qquad (7.2)$$

H : Burial depth to top of pipe, m

γ_S : Soil density, kN/m^3

ii. Live load, kPa (W_L)

The live load is imposed by a source moving over the buried pipe, such as vehicles on a road, railway or load at an airport. The determination of live load is important for shallow burial of less than 3.1 m. The effect of live load decreases as the depth of cover increases. A table of live loads has been developed for highways and railways which are given in the Table 7.2.

Table 7.2: Live Loads on Flexible Pipes

Height of Cover (m)	Highway HS-25		Railway E80
	P_L (N/mm^2)	Distribution Width (L_w) (mm)	P_L (N/mm^2)
0.3	0.108	787	N.R
0.6	0.048	1321	0.1824
0.9	0.036	1854	0.1632
1.2	0.024	2388	0.1272
1.5	0.015	2921	0.1152
1.8	0.012	3454	0.1080
2.1	0.011	3988	0..0846
2.4	0.006	4521	0.0768
3.1	N.S	N.S	0.0528
3.6	N.S	N.S	0.0384
4.3	N.S	N.S	0.0288
4.9	N.S	N.S	0.0240
5.5	N.S	N.S	0.0192
6.1	N.S	N.S	0.0144
6.7	N.S	N.S	0.0132
7.3	N.S	N.S	0.0120
7.9	N.S	N.S	0.0096
8.5	N.S	N.S	0.0072
9.1	N.S	N.S	0.0048
10.7	N.S	N.S	N.S
12.2	N.S	N.S	N.S

Source: AASHTO

Notes:
1) Includes impact where required.
2) N.R indicates that the cover height is not recommended.
3) N.S indicates that the load is non-significant.

iii. Deflection lag factor, DL

For initial deflection, deflection lag factor is taken as 1.0. Long term deflection depends on embedment and compaction of the soil. However, for a conservative design, lag factor can be taken as 1.5 for long term condition.

iv. Soil modulus E'

The most commonly recognised values of soil modulus E' are those of Amster Howard of the US Bureau of Reclamation. Howard examined information from known laboratory and field tests and developed a table of average values of E' which is given in Table 7.3 for computation of deflection of buried flexible pipes.

Table 7.3: Average Values of Modulus of Soil Reaction E′ (for Initial Flexible Pipe Deflection)

E' for degree of compaction of bedding				
Soil type pipe bedding material(Unified Soil Classification System)	**Dumped**	**Slight <85% Proctor, <40% relative density**	**Moderate 85-95 % Proctor, 40-70% relative density**	**High, >95% Proctor, >70% relative density**
	KPa	**KPa**	**KPa**	**KPa**
Fine grained soils (LL<50) Soils with medium to no plasticity CL, ML, ML-CL with less than 25% coarse – grained particles	345	1379	2758	6895
Fine grained soils (LL<50) Soils with medium to no plasticity CL, ML, ML-CL with more than 25% coarse – grained particles Coarse grained soils with fines GM, GC, SM, SC containing more than 12% fines	690	2758	6895	13789
Coarse grained soils with little or no fines GW, GP, SW, SP containing less than 12% fines	1379	6895	13790	20684

Crushed rock	6895	20684	20684	20684
Accuracy in terms of percentage of deflections	±2	±2	±1	±0.5

Source: Amster K. Howard, U.S. Bureau of Reclamation, Denver, "Modulus of Soil Reaction (E') Values for Buried Flexible Pipe,"

Note: Values given in Table 7.3 are consistent with field and laboratory data taken over a 20 year period at Utah State University.

v. Bedding constant, K

The bedding constant is a term which accounts for the reactive force imparted from the pipe bedding material when a pipe is installed. The bedding constant is determined from the bedding angle as shown in the figure 7.1. Values of bedding angles and approximate constants are given in Table 7.4. For most installations the bedding constant is assumed to be 0.1.

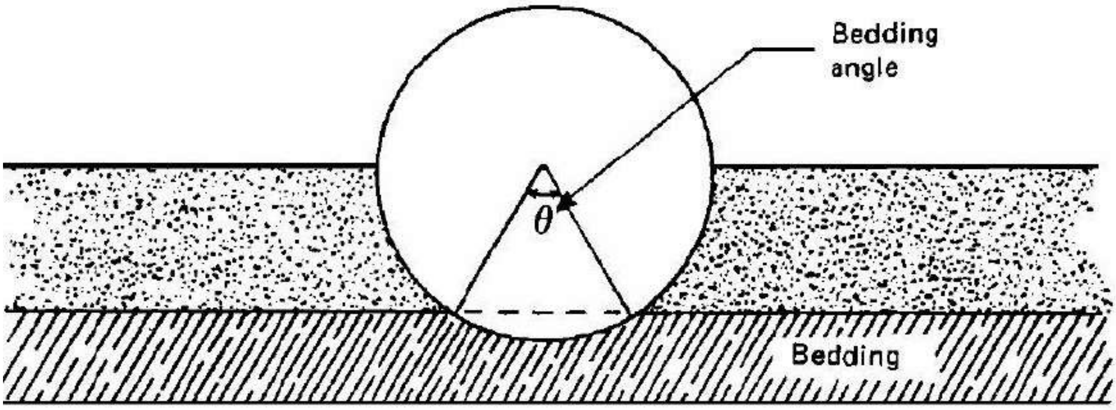


Figure 7.1: Bedding Angle

Table 7.4: Value of constant (K)

Bedding angle in degree	K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

vi. Pipe properties

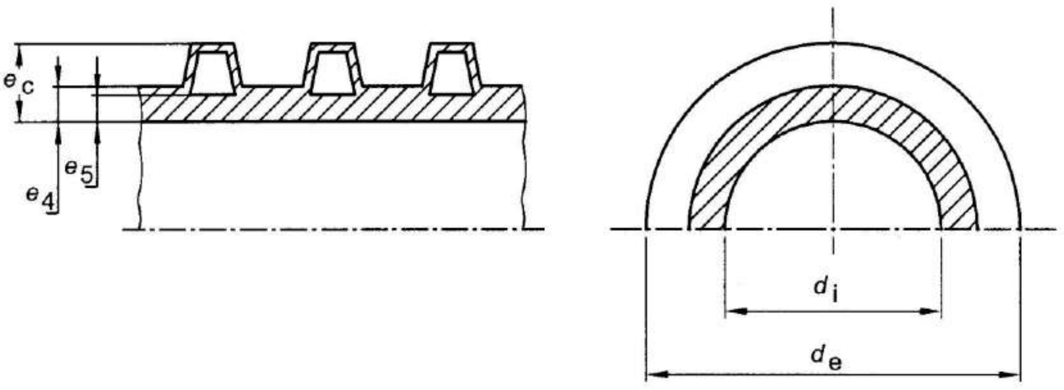
Pipe properties of PE pipes are given in Table 7.5 as per IS code 16098:2013 (Part II).

Table 7.5: Nominal size, Minimum Mean Inside Diameters, Thickness of inside Layers and Socket Length

Sl No.	DN/ID Series		Minimum Wall Thickness		Socket[1)] Length
	DN/ID	$d_{im,Min}$ mm	$e_{4,Min}$ mm	$e_{5,Min}$ mm	A_{Min} mm
(1)	(2)	(3)	(4)	(5)	(6)
i)	75	71	1.0	0.85	27
ii)	100	95	1.0	1.0	32
iii)	125	120	1.2	1.0	38
iv)	135	130	1.2	1.0	39
v)	150	145	1.3	1.0	43
vi)	170	165	1.4	1.0	48
vii)	200	195	1.5	1.1	54
viii)	225	220	1.7	1.4	55
ix)	250	245	1.8	1.5	59
x)	300	294	2.0	1.7	64
xi)	400	392	2.5	2.3	74
xii)	500	490	3.0	3.0	85
xiii)	600	588	3.5	3.5	96
xiv)	800	785	4.5	4.5	118
xv)	1000	985	5.0	5.0	140
xvi)	1200	1185	5.0	5.0	162

1) For selection of A_{min} requirements for socket, refer to the pipe material and construction. For pipe longer than 6 m it is recommended that one produce a larger A_{min} that is specified in this table

Where,
D_{im}, Min : Minimum mean inside diameter of a socket



Where,
e_4 — wall thickness of the inside layer (waterway wall thickness)
e_c — construction height
d_e — outside diameter
d_i — inside diameter

Note:

i. The internal diameter (DN/ID) shall be as per Table 7.5. Other nominal sizes, falling within the range of Table 7.5 are also permitted. For DN/IDs not specified in Table 7.5, the minimum inside diameter, d_{im}, Min, shall be linearly interpolated between the adjacent values specified in Table 7.5.

ii. Values of pipe properties should be obtained from the manufacturer for specific type of flexible pipe.

Table 7.6: Physical properties of PE pipe

Sl. No.	Characteristics	PE
1	Flexural Modulus, E_{min} in MPa	1000-1200
2	Density, kg/m^3	950
3	Poisson's ratio	0.4

Source: Adapted from IS code IS 16098:2013 (Part II)

Note: For long term condition, flexural modulus may be assumes as 30% of the initial modulus as given in the Table 7.6 for purpose of design.

iii. **Pipe stiffness(PS)**

Pipe stiffness value should be provided by pipe manufacturer or can be determined theoretically by the following equation.

$$PS = 53.69 \times SN \qquad (7.3)$$

Where,

PS = Pipe Stiffness in KP_a
SN = Ring Stiffness, KPa

7.2.2.2 Buckling Pressure

Buckling of pipe is a localized failure of the pipe wall structure which is due to insufficient pipe stiffness, deep burial with high ground water table, internal vacuum or poor backfill condition. Therefore, the pipe should be checked for critical buckling pressure against actual buckling pressure giving a factor of safety greater than or equal to 2.

Factor of safety (FS) = Critical Buckling Pressure / Actual Buckling Pressure

a. **Critical Buckling Pressure:** Critical Buckling pressure is calculated by the following formula:

$P_{cr} = 0.6(EI/D^3)^{0.33}(E')^{0.67}$ (7.4)

Where,

P_{cr} : Critical Buckling Pressure, kPa
EI/D^3 = Ring stiffness (SN) of pipe
E' = Average Values of Modulus of Soil Reaction E′

The long and short term values of SN are used to calculate P_{crl} and P_{crs} respectively. (For metal pipes, the long term and short term modulii are identical)

b. **Actual Buckling Pressure:** Actual Buckling pressure is calculated by the following formula:

$P_v = 0.00981\{(R_w H \gamma_s + \gamma_w H_w\} + \frac{1000W_L}{OD}$ (7.5)

Where,

P_v : Actual buckling pressure, kPa
R_w : Water buoyancy factor = 1-0.33 $\left[\frac{H_w}{H}\right]$
H : Burial depth to top of pipe, m
γ_w: Unit weight of water, 1000 Kg/m^3
H_w: Height of ground water over top of pipe, m
W_L : Live load, N linear mm of pipe = OD × P_L
OD : Outside diameter of pipe in mm
γ_s : Soil density Kg/m^3

The following example shall elucidate the application of the above mentioned formulae in designing and checking the flexible pipe installation and its stability.

Illustrative Example 7.1

400 mm (SN 8) inside diameter and 480 mm outside diameter corrugated polyethylene pipe is to be installed having a minimum cover of 0.5 m. Ground water is below the invert of the pipe. Backfill materials are native soil compacted to 85-95% SPD. The density of the backfill material is 15 kN/m^3. Check whether the pipe shall be structurally stable under the aforesaid installation conditions?

Solution:

<u>Check for Deflection:</u>

$$\frac{\Delta y}{D_M} = \frac{K(D_L W_c + W_L)}{\left(\frac{8EI}{D^3}\right) + (0.061 \text{ x } E')} \times 100$$

Where,

K = 0.1

D_L = For initial deflection = 1.0 and for long term deflection = 1.5

W_c = $\boldsymbol{\gamma}_s$ × H = 15 × 0.5 = 7.5 kPa

W_L = 68 kPa as per Table 7.2 (by interpolation)

$\frac{EI}{D^3}$ = 8 kPa (Value of SN by manufacturer)

E' = 6895 kPa from Table 7.2

For short term condition

$$\frac{\Delta y}{D_M} = \frac{0.1(1 \times 7.5 + 68)}{(8 \times 8) + (0.061 \text{ x } 6895)} \times 100 = 1.56\ \%$$

Deflection (%) = 1.56 %

For long term condition

$$\frac{\Delta y}{D_M} = \frac{0.1(1.5 \times 7.5 + 68)}{(8 \times 0.3 \times 8) + (0.061 \text{ x } 6895)} \times 100 = 1.8\ \%$$

<u>Check for Buckling</u>

a. Critical Buckling Pressure

$P_{cr} = 0.6(EI/D^3)^{0.33}(E')^{0.67}$

$P_{crs} = 0.6(8)^{0.33}(6895)^{0.67} = 444.6$ k Pa

$P_{crl} = 0.6(2.4)^{0.33}(6895)^{0.67} = 298.83$ kPa

b. Actual Buckling Pressure

R_w = 1-0.33 $\left[\frac{H_w}{H}\right]$ = 1-0.33 $\left[\frac{0}{0.3}\right] = 1$

$$P_v = 0.00981\{(R_w H\gamma_s + \gamma_w H_w\} + \frac{1000W_L}{OD}$$

$$P_v = 0.00981\{(1 \times 0.5 \times 1500 + 1000 \times 0\} + \frac{0.068 \times 480 \times 1000}{480}$$

$$P_v = 7.35 + 68 = 75.36$$

FS = P_{crl}/P_v = 298.83/75.36 = 3.96 > 2

Hence, Design is Safe

FS = P_{crs}/P_v = 444.6/75.36 = 5.89 > 2

Hence, Design is Safe

CHAPTER – 8: STORM WATER PUMPING

8.1 General

Urban storm water drainage system may encounter situations where gravity flow conditions may not be feasible either due to topographical configuration of low lying or tidal areas and also where the water level of receiving water bodies is higher than the water level of the outfall. Pumping of storm water becomes an imperative need to avoid flooding and water logging of the area under such situations.

8.2 Planning of Pumping Station

Planning of pump station presents the designer with a challenge to provide a cost-effective drainage system that meets the need of the project. Several important considerations are involved in planning and site selection for pump station. The easy access necessary for safe operation, maintenance, and emergency functions must be available at all times. Hydraulic conditions will have primary importance in site selection, but site appearance and sound attenuation should be also assessed. In normal circumstances, the location of the pump station is usually at the drainage system outlet.
Foundation investigations are necessary and enough space must be provided in the area outside the station to accommodate parking as well as movements of large machinery. A dependable energy source is essential. The primary source of electrical power for most storm water pump stations is a public utility. Underground service is preferred for safety and aesthetic reasons, and overhead lines into the station should be avoided, as they present potential safety hazards during large equipment operation.

The essential components that require to be considered in the preparation of the layout for the pumping station are as follows:

- Location of the pumping point
- Pump sump
- Storage reservoir
- Power Source
- Electrical & Mechanical equipment
- Access to site
- Environment quality
- Aesthetics.

8.2.1 Location of the Pumping Point

Location of pumping station should be selected on dry ground free from flooding risk. However, in cases where pumping location lies in low point / flood prone area on account of topographic consideration, the pumping station floor on which the electrical equipment and related accessories are placed should be at higher elevation.

8.2.2 Pump sump

Space for pump and sump should be assessed either for dry wet pump which are having separate sump or Wet well pump which contains the sump inside the pump house.

8.2.3 Storage Reservoir

Storage may be necessary component where reduction of peak flow is desired considering downstream drainage system or receiving bodies of water. In such cases adequate land area may be required.

8.2.4 Power Source

Electrical power supply source from electrical transmission grids should be the best economical option. Transmission grid should be as near as possible to the pumping station to avoid quite high cost involvement in obtaining the power supply from the distant grid.

8.2.5 Electrical & Mechanical Equipment

The pump house should have adequate space to house electrical and mechanical equipment such as switchboard, control panels, transformer, generator room etc. As per Indian Electricity rules, space requirement for these is given as below:

(a) Sufficient space should be available in the pump house to locate the pump, motor, valves, piping, control panels and cable trays in a rational manner with easy access and with sufficient space around each equipment for the maintenance and repairs.
The minimum space between two adjoining pumps or motors should be 0.6 m for small and medium units and 1 m for large units.

(b) Space for control panels should be planned as per the Indian Electricity (I.E) Rules. As per these:

(i) A clear space of not less than 915 mm in width shall be provided in front of the switch board, In case of large panels, a draw out space for the circuit breakers may exceed 915 mm. In such cases the recommendations of the manufactured should be followed.

(ii) If there are any attachments or bare connections at the back of the switch board, the space, if any behind the switch-board shall be either less than 230 mm or more than 750 mm in width measured from the farthest part of any attachment or conductor,

(iii) If the switch board exceeds 760 mm in width, there shall be a passage way from either end of the switch-board clear to a height of 1830 mm,

(c) A service bay should be provided in the station with such space that the largest equipment can be accommodated there for overhauling and repairs.

(d) A ramp or a loading and unloading bay should be provided. In large installations the floors should be so planned that all piping and valves can be laid on the lower floor and the upper floor should permit free movement.

(e) Head room and material handling tackle:

(i) In the case of vertical pipes with hollow shafts motors, the clearance should be adequate to lift the motor clear off the face of the coupling and also carry the motor to the service bay without interference with any other apparatus. The clearance should also be adequate to dismantle and lift the largest column assembly.

(ii) In the case of horizontal pumps (or vertical pumps with solid shaft motors) the head room should permit transport of the motor above the other apparatus with adequate clearance.

(iii) The mounting level of the lifting tackle should be decided considering the above needs and the need of the head room for the maintenance and repair of the lifting tackle itself.

(iv) The traverse of the lifting tackle should cover all bays and all apparatus.

(v) The rated capacity of the lifting tackle should be adequate for the maximum weight to be handled at any time.

8.2.6 Access to site

There should be easy access to the pump station for heavy vehicles carrying machineries, hoisting equipment etc., that are likely to be used during construction and maintenance. Sufficient space should be provided for service road, off street parking, station loading area, turn around area, heavy lifting equipment, road side warning signs including above stated land requirement.

8.2.7 Aesthetics of pumping station

Typical low cost measure to enhance visual quality should be employed

- Allowing adequate area of natural and planted vegetation
- Enclosing unsightly objects such as storage tank etc.
- Using submersible pumps to reduce the size of required above ground facilities
- Using local building materials that blend in with surrounding architecture.
- Providing underground utilities (power supply, phone lines etc).

8.2.8 Environmental Quality

Following environmental qualities may be considered.

8.2.8.1 Air Quality

Diesel generators or engine driven pumps are potential air quality polluters that may be replaced by natural gas or purely grid supplied electrical energy.

8.2.8.2 Noise

Noise attenuation is a necessary concern near residential areas. Wherever practicable one or more of the following measures may be adopted:

a) Use submersible pump
b) Where submersible pumps are not practicable, use electrically driven motor if engine is used, provide mufflers.
c) Build pump house from concrete or masonry.
d) Sound insulation of pump house wall may be an option.

Environmental audit should be carried out at regular intervals.

8.3 Design of Pumping Station

8.3.1 Type of pump stations

Basically two types of pump stations are constructed for the purpose of storm water pumping viz, wet pit and dry pit for which typical diagrams are illustrated in the following figures.

8.3.1.1 Wet pit pump station

In wet pit system, the pump is either submerged under water connected with a drive shaft to an overhead electrical motor or the submerged pump is directly coupled with submersible motor as shown in Fig 8.1 & Fig 8.2.

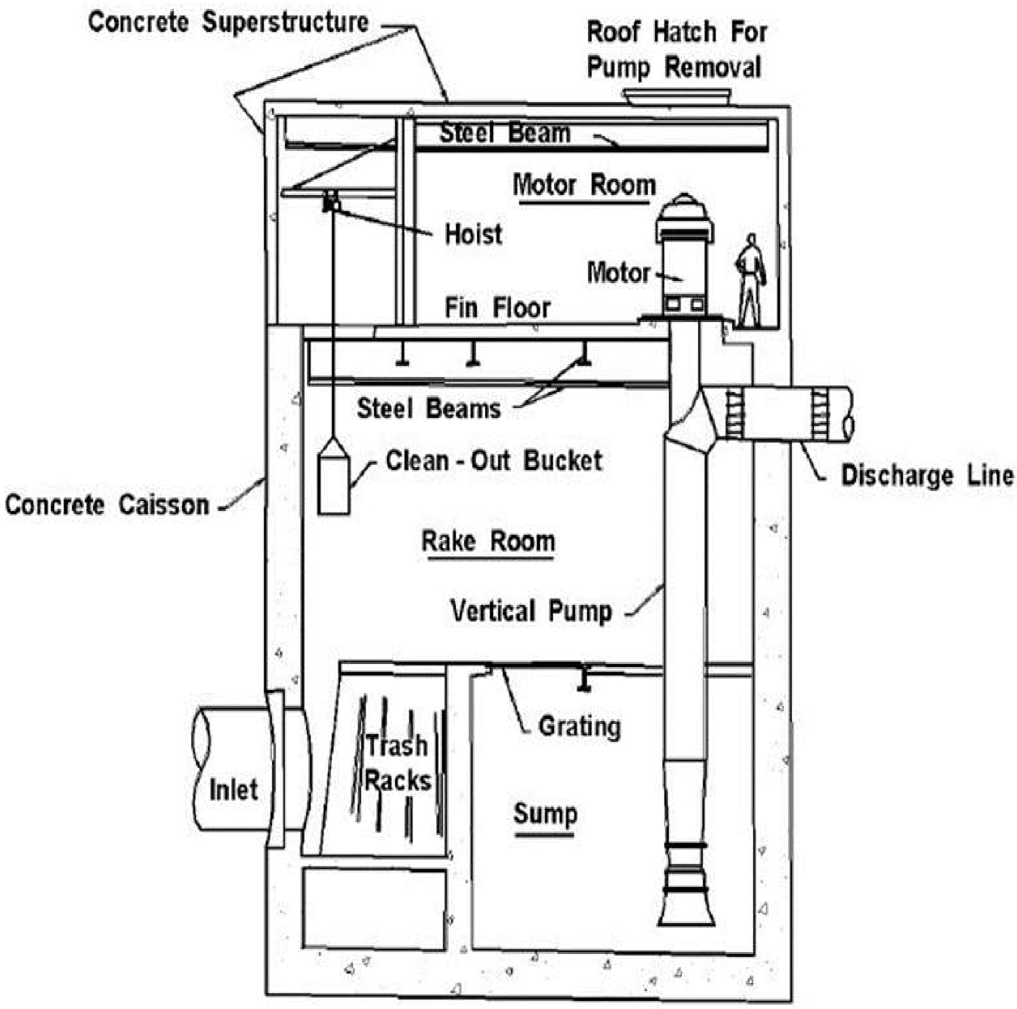


Figure 8.1: Wet Pit Pump Station with vertical turbine pump

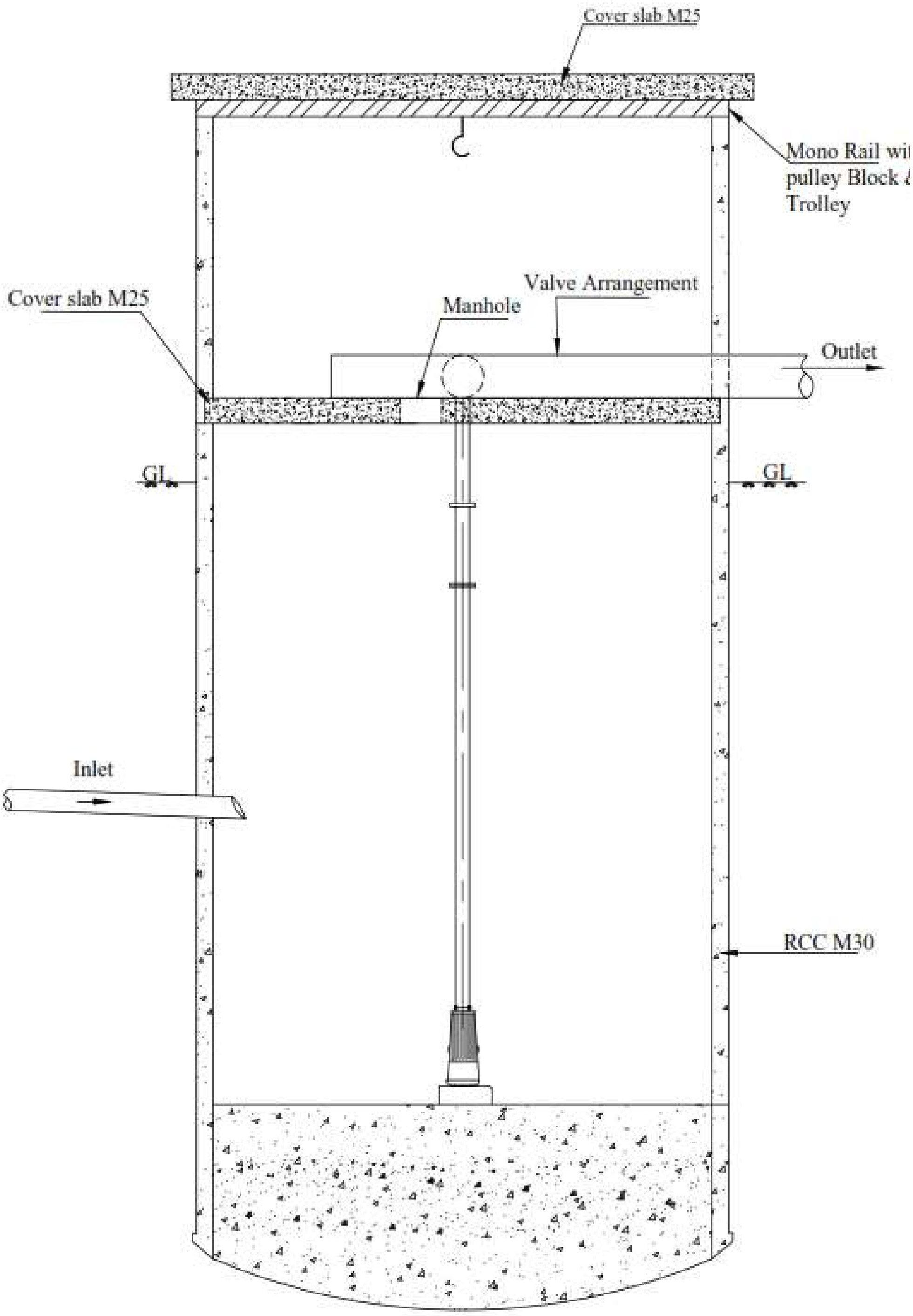


Figure 8.2: Wet pit pump station with submersible pump

8.3.1.2 Dry pit pump station

In dry pit system, the horizontal centrifugal pump directly coupled with motor are installed on the floor of the dry pit with its suction pipe connected to the sump as shown in the figure 8.3.

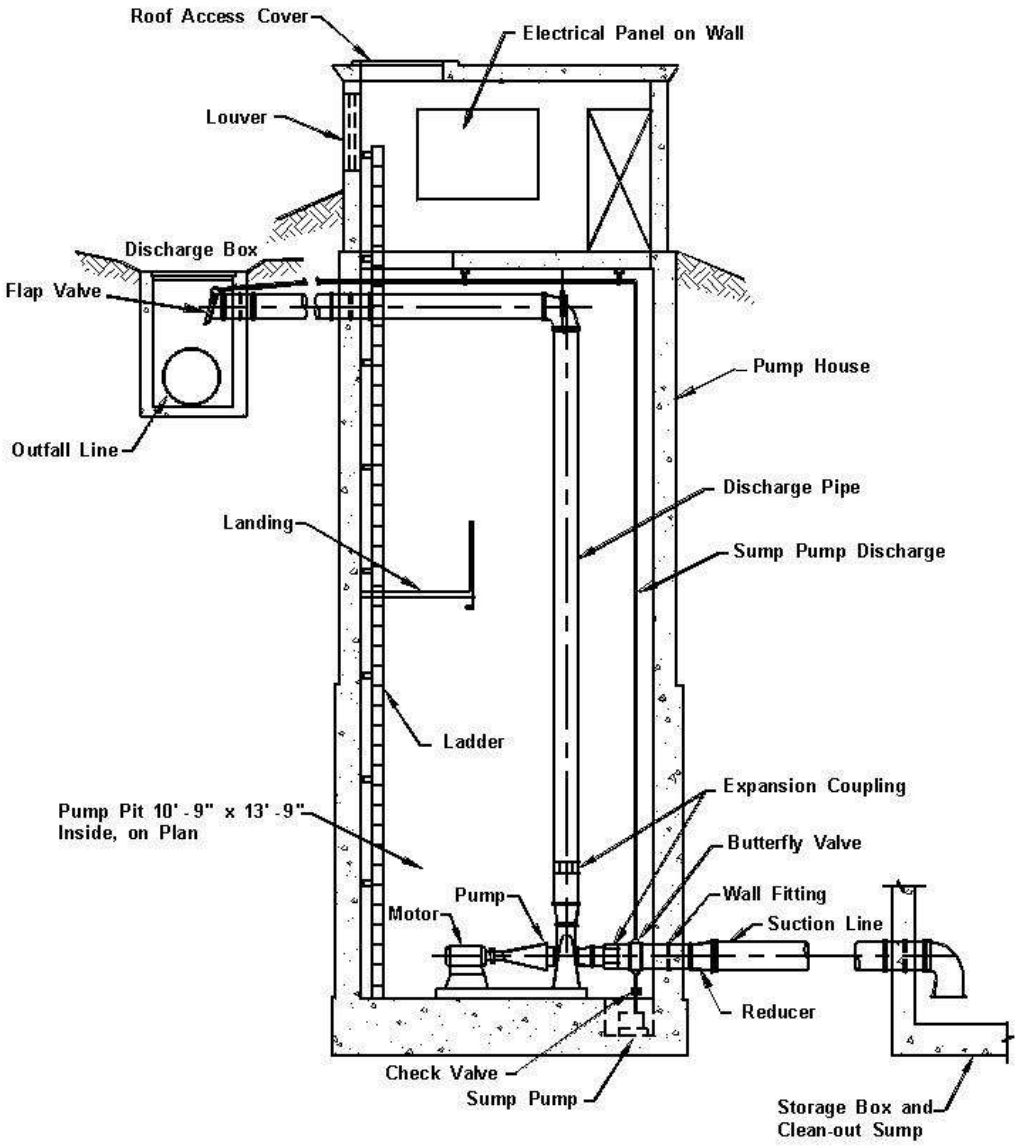


Figure 8.3: Dry Pit Pump Station with horizontal centrifugal pump

8.3.1.3 Trash Screens

Trash screen should be provided at the entrance to wet pit if large debris is anticipated in the incoming storm water. For storm water pumping station simple steel bar screens are adequate. Usually bar screens are inclined with bar spacing approximately 58.0 mm. constructing the screen in modules facilitate removal for maintenance. If the screen is relatively small an emergency overflow should be provided against clogging and subsequent surcharging the collection system.

In case substantial amount of sediments are anticipated a catch basin may be provided to settle out the solids.

8.3.2 Choice of Pump Type

The choice of pump type may be considered from following type of centrifugal pumps that are commonly used in storm water pumping in accordance with the criteria of requirement.

1) Axial flow pump

Axial pumps deliver water parallel to the pump axis and drive shaft.
They are generally used for low head and high discharge application. These pumps cannot handle debris because the propeller may be damaged if they strike large and hard object.

2) Radial flow pumps

Radial flow pumps use centrifugal force to deliver water. They can handle wide range of head and discharge, the best for high head applications. They can handle debris quite well. A single vane non clog impeller handles debris the best as it provides largest impeller opening.

3) Mixed flow pumps

Mixed flow pumps are similar to axial flow pumps except that they create pressure and velocity to liquids by the centrifugal force of impellers and the lifting force of vanes and thus they are combination of the above two types. They are suitable for intermediate head and discharge application with better ease in debris handling.

All pumps can be driven either with directly coupled or through a driving shaft with electrical motors or engines. In case of submersible pumps, submersible electrical motor are coupled with the pumps submerged under storm water. Submersible pumps have advantages in simplifying the design, construction maintenance and thereby reducing cost of the pump station.

8.3.3 Design of wet pit

When automatic controls and variable discharge pump are installed, the wet well should have storage of 5 min detention of minimum inflow within the wet well. In absence of such control and variable discharge pump, the wet well should have a storage of a minimum cycle time of peak flow or should have a storage from 10 min to 15 min of incoming peak flow calculated by the following formula:

$$V = Q \times t \quad (8.1)$$

Where,

V = Volume of wet well in m^3
Q = Peak discharge in m^3/min
t = Cycle time in minutes

However, the minimum allowable cycle time ‘t’ is designated by the pump manufacturer based on electric motor size.

Minimum 3 duty pumps may be installed in the pump house for peak flow, half peak flow and quarter peak flow to be operated singly or in combination as required consistent with variable inflow. Stand by units should be provided either 50% or 100% of the duty pump units as decided by design engineer. The internal diameter of the well shall be kept such that number of submersible pumps coupled with motors or shaft driven installed inside the well may not create mutual interference with each other. The data required as such should be provided by pump manufacturer along with specified submergence depth of pump for functional efficiency.

In case reduced constant discharge is required, dry pit pump station should be used. The pumps should be connected through a suitable penstock with the storage tank, water level sensor viz. float switch, electronic probes, and ultrasonic devices etc. should be used to control the pumping system.

8.3.4 Design Capacity of storage tank

The rate of pumped discharge of the storm water should be fixed as per peak inflow of storm water. In case attenuation in peak runoff of the catchment to down steam conveyance main or receiving water bodies is required, the storage volume required to detain the surplus water in a storm water tank.

An example has been given herein to reduce the peak flow of 0.62 m^3/sec to 0.40 m^3/sec by providing a designed capacity of storage tank.
Capacity of the storm water storage tank to accomplish the reduction of peak flow discharge either to downstream facilities or to receiving water bodies can be achieved by operating the outflow hydrograph over inflow hydrograph for the design rainfall event over watershed contributing to the tank. The estimated storage volume shall be area of shaded portion (intercepted between outflow hydrograph and the inflow curve as shown in the figure 8.4 below.

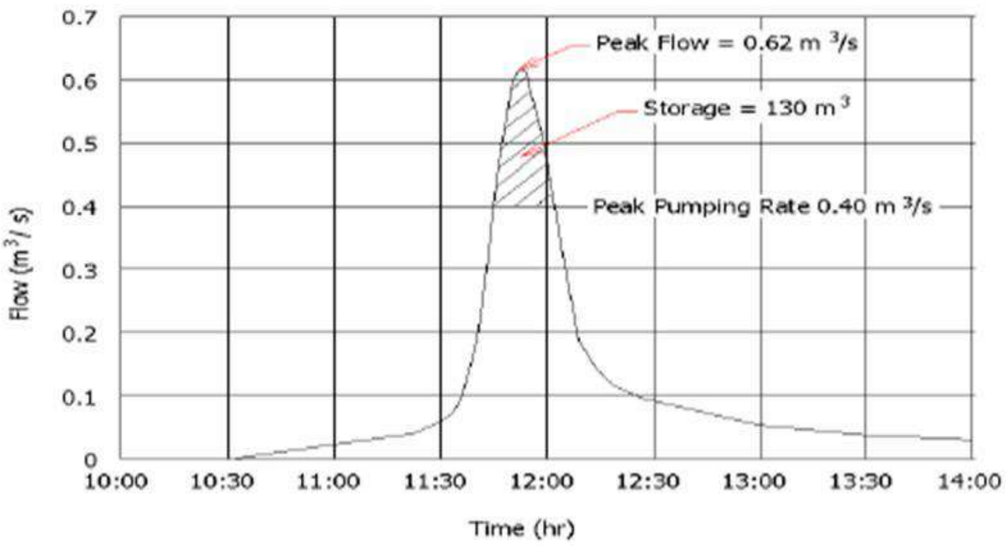


Figure 8.4: Inflow Hydrograph

By the above graph, it could be easily inferred that the peak flow can be reduced to 0.40 m^3/sec by providing a storage tank of 130 m^3 capacity.

8.3.5 Pump Characteristic Curves

There are two types of pump curves i.e. system curve and pump curve as described below:

8.3.5.1 System Curve

A plot of curve for evaluated values of head to be developed by the pump against different values of flow rates is called the system head curve. The total dynamic head comprises total of the following:

Total Dynamic Head = $H_s + H_f + H_v + H_l$ (8.2)

Where,
$H_s =$ Static head measured from liquid level of the sump to the delivery point (m)
$H_f =$ Frictional head in total pipe length from foot valve to delivery end (m) .
$H_v =$Velocity head $\frac{v^2}{2g}$ (m)
$H_l =$ Head loss in fittings and valves (m)

Residual head may also be added if required (m)

8.3.5.2 Pump curve

Pump curve is a plot of curve of pump flow rates versus various heads. Pump curves are supplied by manufacturer of the pump. The point of intersection of pump curve and system curve as shown in the figure 8.5, 8.6, 8.7 is called the operating point or design point of the pump.

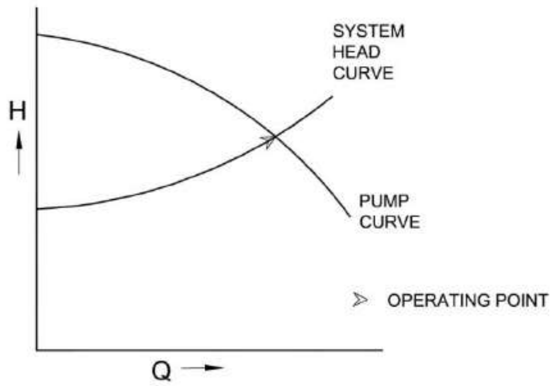


Figure 8.5: Operating point of the curve

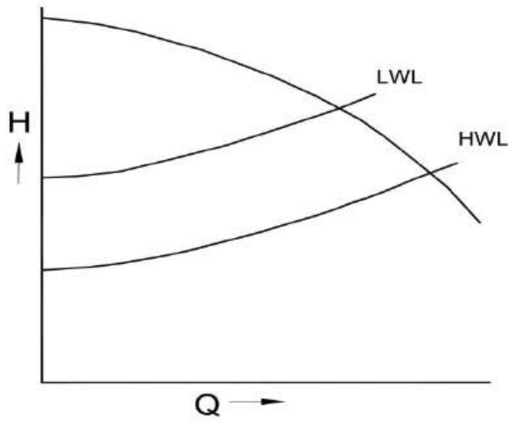


Figure 8.6: Change in Operating Point of Pump with change in Water level in Suction Sump

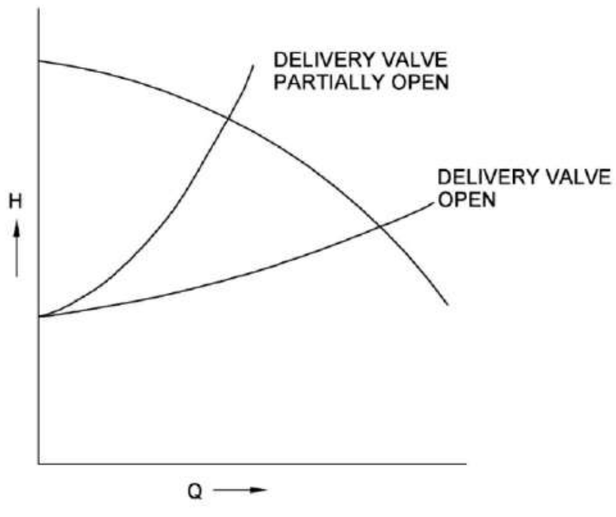


Figure 8.7: Change in operating point of pump by operation of delivery valve

8.3.6 Net Positive Suction Head Required (NPSHr)

The suction lift capacity of a pump depends upon its NPSHr characteristics. The meaning of NPSHr can be explained by considering an installation of a pump working under suction lift as illustrated in Fig 8.8.

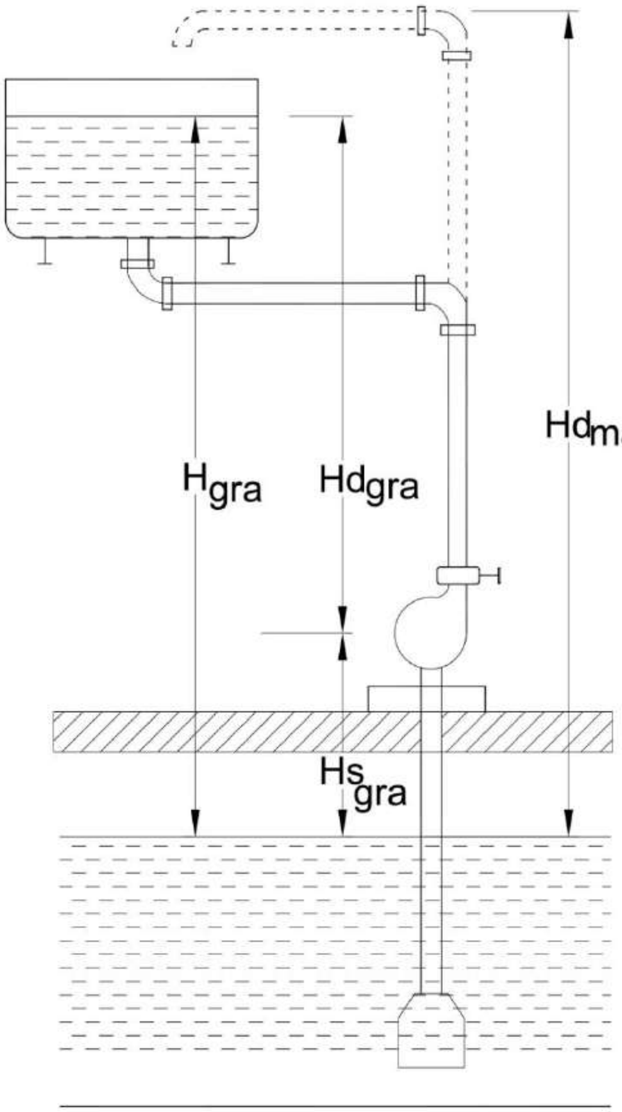


Figure 8.8: Schematic Representation of NPSHr

When a pump, installed as shown is primed and started, it throws away the priming water and has vacuum developed at its suction. The atmospheric pressure acting on the water in the suction sump then pushes the water through the foot valve, into the suction line, raising it upto suction of the pump. While reaching upto the suction of the pump, the energy content of the water, which was one atmosphere when it was pushed through the foot valve would have reduced, partly in overcoming the friction through the foot valve and the piping and the pipe fittings, partly in achieving the kinetic energy appropriate to the velocity in the suction pipe and partly in rising up the static suction lift. The energy content left over in the water at the suction face of the pump is thus less than one atmosphere until here the flow is a fairly streamlined flow. But with the impeller rotating at the pump suction, the flow suffers turbulences and shocks and will have to lose more energy in the process. This tax on the energy of the water demanded by the pump, before the pump would impart its energy, is called the NPSHr of the pump.

The NPSHr characteristics of a pump are parabolic, increasing with flow rate. Pumps of high specific speed have high NPSHr.

8.3.6.1 Vapour Pressure and Cavitation

The energy of the water at the pump suction, even after deducting the NPSHr should be more than the vapour pressure VP, corresponding to the pumping temperature. The vapour pressures in meters of water column (mWC), for water at different temperatures in degrees Celsius are given in Table 8.1.

Table 8.1: Vapour Pressure of Water

^{0}C	(mWC)
0	0.054
5	0.092
10	0.125
15	0.177
20	0.238
25	0.329
30	0.427
35	0.579
40	0.762
45	1.006
50	1.281

If the energy of the water at the pump suction would be less than the vapour pressure, the water would tend to evaporate. Vapour bubbles so formed will travel entrained in the flow until they collapse. This phenomenon is known as cavitation. In badly devised pumping systems, cavitation can cause extensive damage due to cavitation erosion or due to the vibration and noise associated with the collapsing of the vapour bubbles.

8.3.6.2 Calculating Net Positive Suction Head Available (NPSHa)

To ensure against cavitation, the pumping system has to be so devised that the water at the pump suction will have adequate energy. Providing for this is called as providing adequate Net Positive Suction Head available (NPSHa). The formula for NPSHa hence becomes as follows.
NPSHa= Pressure on the water in the suction sump

$$= P_s - Hf_s - \frac{Vs^2}{2g} - Z_s - V_P \qquad (8.3)$$

Where,

P_S = suction pressure
Hf_s = friction losses across the foot valve, piping and pipe fittings

V_S = Velocity-head at the suction face
Z_s = Potential energy corresponding to the difference between the levels of the pump-centre line and of the water in the suction-pump
V_P = Vapour pressure

While calculating NPSHa, the atmospheric pressure at the site should be considered, as the atmospheric pressure is influenced by the altitude of the place from the mean sea level (MSL). Data on the atmospheric pressure in mWC for different altitudes from MSI, is given in Table 8.2.

Table 8.2: Atmospheric Pressure in mWC at different altitudes above MSL

Altitude above MSL in m	(mWC)
Upto 500	10.3
1000	9.8
1500	9.3
2000	8.8
2500	8.3
3000	7.8
3500	7.3
4000	6.8

8.3.6.3 Guidelines on NPSHr

The NPSHa has to be so provided in the systems that it would be higher than the NPSHr of the pump. The characteristics of the pump's NPSHr are to be obtained from the pump-manufacturers.

8.3.7 Electrical Motors (prime mover)

Generally either squirrel cage motors or slip ring motors are used as prime movers for pump-drive as per requirement of load i.e up to 20 B.K.W load squirrel cage motors can be used above to that slip ring motors are used.

8.3.7.1 Motor Rating

Power required at pump shaft to deliver required quantity of fluid to a specified lift (head, measured in height of water column).

B.K.W. (brake Kilo Watt) = $\frac{Q*S*H}{102*e*E}$ (8.4)

Where,
Q= in L.P.S (pump required discharge)
H= height in meter (lift required)
S = specific gravity of fluid
e = Efficiency of motor
E= Efficiency of pump

A multiplying factor over the computed B.K.W should be applied as shown in the table 8.3 below as well as for losses in bearing etc., additional 3 kW should be added to arrive at the required rating of the motor.

Table 8.3: Multiplying factor for motor rating

Required BKW of the pump	Multiplying factor to decide motor rating
Up to 1.5	1.5
1.5 to 3.7	1.4
3.7 to7.5	1.3
7.5 to 15	1.2
15 to 75	1.15
Above 75	1.1

8.3.7.2 Voltage rating

General guidance on the standard voltages and corresponding range of motor ratings are given in Table 8.4.

For motor of ratings, 225 kW and above, where high tension (HT) voltages of 3.3 kV, 6.6 kV and 11 kV can be chosen, the choice should be made by working out relative economics of investments and running costs, taking into consideration cost of transformer, motor, switchgears, cables etc.

Table 8.4: Selection of motor based on supply voltage

Supply	Voltage	Range of Motor rating in KW	
		Minimum	Maximum
Single phase A/C	230 V	0.3	2.5
Three phase A/C	415 V		250
Do	3.3 KV	225	750
Do	6.6 KV	400	
Do	11 KV	600	
D.C	230 V		150

Note: When no minimum is given, very small motors are feasible. When no maximum is given, very large motors are feasible.

Table 8.5: Protective enclosure and environment type of motors

Type	Environment type	Description of environment
Screen protected drip proof SPDP	nil	Indoor, dust free environment.
Total enclosed	IP44	Indoor dust prone areas
Total enclosed fan cooled TEFC	IP54	Normal out door

Type	Environment type	Description of environment
	IP 55	Outdoor at places of heavy rainfall

8.3.8 Transformer Substation

Normally outdoor substations are provided. However on considerations of public safety and for protection from exposure to environmental pollution, the substations may be indoors

i. Lightning arresters
ii. Gang operated disconnectors (GOD) are provided in outdoor substation. In indoor substation, circuit breakers are provided. In case of outdoor substations of capacities 1000KVA and above, circuit breakers should be provided in addition to GOD.
iii. Drop out fuses for small out door substations.
iv. Overhead bus bars and insulators.
v. Transformer.
vi. Current transformer and potential transformer for power measurement.
vii. Current transformers and potential transformers for protection in substations of capacity above 1000 KVA.
viii. Fencing.
ix. Earthing

Earthing should be very comprehensive, covering every item in the substation and in accordance with IS: 3043.

8.3.8.1 Transformer Rating

a) Estimation of total power consumption

The first step in the transformer rating is to estimate the power consumption for various units of pumping station such as:

- Power consumed of working motors
- Power consumed by ventilating equipment
- Power consumed by automation equipment
- Power consumed by lighting
- Power consumed by fan etc.

In this manner the total power consumed may be estimated to be ‘A’ KW, then add 10% for miscellaneous consumptions. Thus, say ‘B’ KW is the total power required for which transformer is to be rated.

Total power requirement – 'B' KW
Then transformer rating – 'B' KW /0.85 = KVA
Where, 0.85 is a power factor

Example 8.1:

Total Power consumption = 1000 KW
Hence, transformer KVA required at 0.85 Power Factor and 10% margin
= 1000 × 1.1/0.85 = 1294 KVA
Hence, provide next commercial rating as 1600 KVA

8.3.8.2 Improvement of Power Factor

For improvement of power factor, appropriate capacitors should be provided. Capacitors may be located in the control panel or separately by an automatic power factor correction panel which monitors a bank of capacitors through a power factor sensing relay and appropriate contactors.

8.3.9 Lighting

The interior of pump stations, shall be provided with sufficient lighting system specially designed to achieve best illumination suited to the station layout. Energy efficient fluorescent fixtures are preferred. Lighting shall be at adequate levels for routine service inspections and maintenance activities as given in the Table 8.6.

Table 8.6: Illumination Levels

S No	Area		Illumination Level (LUX)
1	Sub Station Building	:	250
2	Pump House	:	200
3	Control Room	:	300
4	Transformer Yard	:	50
5	Aerator Operating Platform	:	50
6	Screen & Grit Separator area	:	50
7	Chemical Stores	:	100
8	Centrifuge House	:	150
9	Chlorine Tonner Room	:	150
10	All other indoor areas	:	100
11	Outdoor Plant Area	:	20
12	Roads	:	10

8.3.10 Ventilation

Pump stations shall be provided with a separate ventilating system and shall be sized to provide a minimum of 10 air changes per hour. Ventilation systems shall be capable of matching inside air temperature to outside air and shall be automatic. Ventilation shall be accomplished by the introduction of fresh air into the pump station.

8.3.11 Pumping main

A pipe line either long or short when used to transport storm water against gravity under pressure generated by an arrangement of a suitable pumping unit is generally termed as pumping main. The design of such pumping main depends on the following factors:

a. Quantity of design flow
b. Available pressure or head
c. Losses of friction in pipe and fittings
d. Allowable velocity of flow
e. Quality of fluid
f. Material of pipe

Process design for sizing the pipe diameter is given as follows:

a. Determine design flow rate in cum/hr
b. Determine the total head in m
c. Select pipe material capable to withstand design hydraulic pressure
d. Choose allowable self-cleansing velocity not less than 0.6m/sec and not more than 1.4m/sec
e. Use Hazen – William formula or Darcy Weisbach formula as given in the following section
f. Keep friction loss between 0.9m and 1.2m per 1000.0m to maintain prescribed velocities as given in sl(d)
g. Test the pumping main against surge pressure and provide necessary protection if required.

In case a pumping main is considerably long involving heavy investment attempt should be made to design an economical size of pipe. The design of economical size of pipe (diameter) is based on the following parameters:

a) A design horizon of 30 years
b) Different type of pipe materials conducive to site conditions
c) Different sizes of pipes against different hydraulic grades which are considered for given quantity of storm water to be pumped

d) Useful service life of different materials and their relative costs as laid in position
e) The duty, capacity and installed cost of pump sets required against the corresponding sizes of pipe lines under consideration
f) Recurring costs like energy cost and annual maintenance cost of corresponding sizes of pipe lines under consideration
g) Prevailing rate of interest

The process of computation may be referred to Manual on Water supply and Treatment (1991), a publication of MoHUA, Govt. of India.

8.3.11.1 Friction flow formula to size Pumping Mains

Darcy Weisbach and Hazan Williams formulae are given for pressure-pipe frictional assessment.

$$\boldsymbol{h_f = \frac{flv^2}{2gD}} \tag{8.5}$$

Where,
h_f = Head loss in pipe due to friction (m)
f = Darcy Weisbach friction factor can be found from Moody's diagram given in Appendix A 5.6.
L = length in pipe (m)
D = Diameter of pipe (m)
V = Velocity of flow in pipe m/sec
g = Acceleration due to gravity (m/sec^2)

Hazen William Formula

$$Q = 1.292 \times 10^{-5} \times C \times d^{2.63} \times S^{0.54} \tag{8.6}$$
$$V = 4.567 \times 10^{-3} \times C \times d^{0.63} \times S^{0.54} \tag{8.7}$$

Where,
Q = Discharge in m^3/hr
d = pipe diameter in mm
S = Hydraulic Slope
C = H. W. Coefficient can be found from Table 8.7

Table 8.7: Values of Roughness coefficient, C, in Hazen-Williams formula

Material	Hazen-Williams C
New Pipes	
Cast Iron	130-100
Concrete(RCC&*PCC with S/S	150-120

Material	Hazen-Williams C
Concrete-lined Galvanized iron	120
Plastic	150-120
Steel welded joints lined with cement or bituminous enamel	150-120
Asbestos cement	150- 120
Welded Steel	140- 100

Source: Manual on Sewerage and Sewage Treatment Systems, CPHEEO

However, the selection of pipe materials in order to minimize the head losses in pipes should be considered. Besides, slime and sediment deposits on internal surfaces of the pipes do affect the smoothness of internal surfaces of the pipes consequently contributing to frictional losses. Metal pipes are generally provided with lining of smooth material such as PVC etc. in order to reduce the losses. Other factors such as pipe joints, manholes, branch pipes, bends, elbows, sudden enlargement and reduction in pipe sizes, inlets, outlets etc shall also contribute to losses that need to be computed in system design. There will be pressure losses in fittings which shall be accounted for as in Table 8.8 by multiplying the factor with the velocity head.

Table 8.8: Friction factor for fittings in pumping mains

No.	Types of Fittings	Factor
1.	Sudden contraction	0.5
2.	Entrance shape well rounded	0.5
3.	Elbow 90 degrees	1.0
4.	Elbow 45 degrees	0.75
5.	Elbow 22 degrees	0.5
6.	Tee 90 degrees	1.5
7.	Tee in straight pipe	0.3
8.	Gate valve open	0.4
9.	Valve with reducer and increaser	0.5
10.	Globe valve	10.0
11.	Angle	5.0
12.	Swing Check	2.5
13.	Venturimeter	0.3
14.	Orifice	1.0

8.4 Storm water Storage Pond/Basin

The primary function of storm water storage pond is either to store the storm water and gradually release through controlled mechanism to receiving water bodies, conveyance system or completely consumed via infiltration and evaporation. There are basically two types of storm water storage tank as described below.

8.4.1 Detention ponds/Basin

Detention facilities provide temporary storage of storm water that is released through an outlet that controls flows to pre-set levels. Detention facilities typically flatten and spread the inflow hydrograph, lowering the peak to the desired flow rate. It is generally planned to limit the peak out flow rate to that which is existed from the same watershed which is existed before development for a specific range of flood frequencies.

8.4.2 Retention ponds/Basin

Retention ponds may also be called an extended detention pond as defined above as all the stored storm water is .absorbed through infiltration and evaporation over a long period of time. Nevertheless the stored water may be used if need be for water supply and recreational purposes etc. Pervious bottom should be provided in these tanks to ensure sufficient infiltration capability to empty the pond within a reasonable time. This is discussed in rain water harvesting section.

8.4.3 Site Selection

Proximity to flood prone area may be a primary consideration while selecting site for detention basin. The nearer the site to such areas, the larger the tributary areas that could be controlled by the site.

8.4.4 Adequacy of the site size

Land should be available of adequate size as determined by areal extent of the site such that required volume of water could be stored temporarily on site.

8.4.5 Topographic consideration

Topographic configuration should permit gravity driven inflow and out flow from the detention basin which is the most desirable situation in locating the site for detention basin.

8.4.6 Access to the site

Access must be provided for inspection and maintenance either from adjacent publically owned land or through private owned land under access easement provision.

8.4.7 Design of storm water Storage Pond/Basin

The final design computation for detention basin/pond requires three curves:

- An inflow hydrograph for design rainfall events occurring over the catchment contributing to the basin/pond.
- A stage versus storage curve
- A stage versus discharge curve

A preliminary estimate of the storage required to reduce the peak flow within desired limits to be released to downstream facilities or receiving bodies of water should first be computed by the method described as follows:

- Obtain an inflow hydrograph for the design rainfall event occurring over the catchment contributing to the pond
- Develop an approximate out flow hydrograph either by a straight line or by sketching an assumed out flow of the same time base as that of inflow hydrograph. Peak flow should be kept below inflow hydrograph peak to the desired level.
- Operate the above out flow hydrograph by super imposing on the inflow hydrograph as shown in the figure 8.9
- Area of intercepted portion (shaded) within two hydrographs in the figure 8.9 shall give the initial storage requirement of the detention pond.

8.4.8 Optimization of Detention Tank/ Basin Capacity

To optimize the tank capacity 'design storm' draining the given catchment are routed through the basin to determine the maximum storage volume and water level in the basin corresponding to the maximum allowable out flow rate. A number of trials may have to be worked out to maximize tank volume. However manual calculations for the number of hydrographs that need to be estimated and routed through the tank will be too tedious, complex and time consuming. A suitable computer model may be used to perform these calculations with ease and promptness.

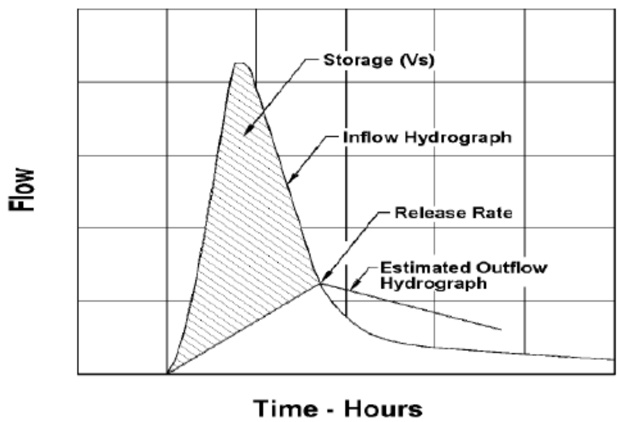


Figure 8.9: Inflow and Outflow Hydrograph

8.4.9 Basin outlet

Suitable outlets are provided for planned release of water from a detention basin. A stage-discharge relation curve is developed for full range of flows that the outlet structure would experience. Weir or orifice is typically provided as outlet device consistent with high and low rate of discharge respectively. These outlet arrangements are generally housed in a riser structure connected to a single outlet conduit that passes through the basin embankment and discharges to the point of interest. Care should be taken to ensure that stage- discharge relationship adequately reflects the range of different flow regimes that the structure will operate upon.

CHAPTER 9: INTEGRATION OF RAINWATER HARVESTING IN STORM WATER DRAINAGE DESIGN

9.1 General

The conventional approach of storm water drainage systems design is considered to be uneconomical and infeasible in many cities due to its current pace of sprawling. The ever increasing urban runoff primarily due to growing size of urban areas, rampant paving of urban spaces and global warming induced climate change have posed new challenges before city authorities. In order to address these issues and to control the runoff at source itself, integrating rain water harvesting (RWH) systems in storm water drainage systems design is increasingly being adopted.

9.2 Integration of rainwater harvesting & recharge systems with storm water drainage design

With increase in number of large urban sprawling, the problems due to frequent flooding and consequent damages of life and property have posed a difficult challenge before city managers. Often due to huge cost involvement and lack of space to accommodate wide storm water drain sections, city planners and engineers are emphasizing integration of rainwater harvesting & recharge systems in storm water drainage systems design. The various options / techniques for rainwater harvesting and recharge, suitable for integration with storm water drainage system design, are listed below:

9.2.1 Rainwater harvesting techniques

To minimize the quantity of storm runoff reaching to the storm water drains, a certain quantity of storm water can be stored / percolated by introducing suitable techniques as below:

I. In-situ storage / percolation within or around premises
II. Storage of runoff in nearby pond / water tank
III. Percolation of storm water inside / outside the drains along its stretch
IV. Spreading water for recharge in low lying areas and park / gardens etc.
V. Disposal to reservoir / water body

9.2.2 In-situ storage / percolation

Rain water can be either stored within a building premises or it can be diverted to a suitable place for use as explained below:

9.2.2.1 Roof top rainwater collection potential

Rain water from the roof can be safely collected through rain water pipe either in a constructed underground tank / reservoir for domestic use or can be percolated / recharged in the ground water.

In a city having annual rainfall of 1000 mm, a roof top of an area of 100 sqm has potential to collect rain water to the tune of 1,00,000 ltr (100 m^2 * 1 m) in a given year. At rate of 100 lpcd consumption per person for a family of 4, the water can be sufficient to meet various domestic requirements for (1,00,000 / 400 = 250 days). However, it is to be stressed that before the onset of the monsoon season, the roof surface should be properly cleaned.

In places where there are constraints in storing the rainwater, the same can be safely recharged into the ground through various recharge techniques like percolation pits, abandoned tube wells etc. Even partial storing / recharging of rain water will go a long way in reducing peak runoff in storm water drains, thereby, reducing the incidences of flooding in low lying area. Same approach can be followed on community basis as well. However, due care should be taken to ensure that polluted water is not allowed to enter into the system and the system is periodically cleaned to function as per design requirements.

A typical roof top rainwater harvesting system is shown in Fig 9.1 below.

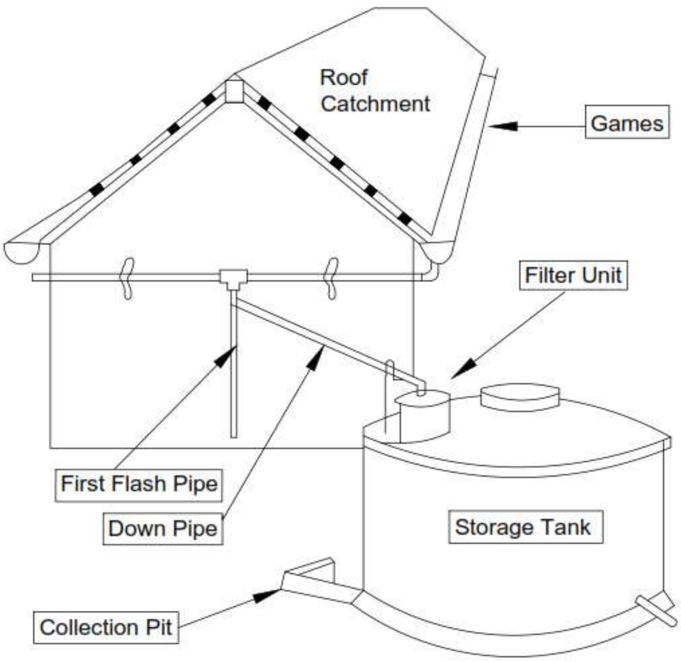


Figure 9.1: Typical Rooftop rainwater harvesting system

The Central Ground Water Board has prepared a ready reckoner to serve for estimating water from roof tops (m^3) and is shown in Table 9.1.

Fig 9.1 shows a typical roof catchment, the gutters fitted at the eaves, down spout connected with the gutter at upper end and with rain water pipe at the lower end. The rain water pipe is fixed on the wall by clamps and joins the storage tank on the ground. The various components of RWH are briefly described below.

1. **Roof Surface:** The roof surface is the area which can be either flat or sloping. This receives the rainfall and from which the collected rainwater is to be harvested.

2. **Gutters**: These are made of different materials such galvanized iron sheet folded to desired shape & size. Semi-circular gutters are prepared from PVC material. Gutters are made 10 to 15 percent over sized according to flow during highest intensity rainfall.

3. **Conduits:** These are pipelines or drain pipes that carry rain water from roof catchments to harvesting system.

4. **Storage tanks:** Such tanks of various sizes to accommodate harvested rain from roof tops can be of varied sizes and placed above or below ground depending upon availability of space. Reinforced cement concrete (RCC) and polyethylene and metal sheets are commonly used to preparing various shapes & sizes of storage tanks.

Table 9.1: Availability of Rainwater through Roof Top Rainwater Harvesting

Roof top area (Sqm.)	Rainfall (mm)											
	100	200	300	400	500	600	800	1000	1200	1400	1600	1800	2000
Harvested Water from Roof Top (cum)													
20	1.6	3.2	4.8	6.4	8	9.6	12.8	16	19.2	22.4	25.6	28.8	32
30	2.4	4.8	7.2	9.6	12	14.4	19.2	24	28.8	33.6	38.4	43.2	48
40	3.2	6.4	9.6	12.8	16	19.2	25.6	32	38.4	44.8	51.2	57.6	64
50	4	8	12	16	20	24	32	40	48	56	64	72	80
60	4.8	9.6	14.4	19.2	24	28.8	38.4	48	57.6	67.2	76.8	86.4	96
70	5.6	11.2	16.8	22.4	28	33.6	44.8	56	67.2	78.4	89.6	100.8	112
80	6.4	12.8	19.2	25.6	32	38.4	51.2	64	76.8	89.6	102.4	115.2	128
90	7.2	14.4	21.6	28.8	36	43.2	57.6	72	86.4	100.8	115.2	129.6	144
100	8	16	24	32	40	48	64	80	96	112	128	144	160
150	12	24	36	48	60	72	96	120	144	168	192	216	240
200	16	32	48	64	80	96	128	160	192	224	256	288	320
250	20	40	60	80	100	120	160	200	240	280	320	360	400
300	24	48	72	96	120	144	192	240	288	336	384	432	480
400	32	64	96	128	160	192	256	320	384	448	512	576	640
500	40	80	120	160	200	240	320	400	480	560	640	720	800
1000	80	160	240	320	400	480	640	800	960	1120	1280	1440	1600
2000	160	320	480	640	800	960	1280	1600	1920	2240	2560	2880	3200
3000	240	480	720	960	1200	1440	1920	2400	2880	3360	3840	4320	4800

The design guidelines of RWH system is as follows:

a) Conveyance System

Conveyance system includes gutters and down pipes ending at common collection chamber. Following recommendations to be followed as per the guidelines for Rainwater Harvesting:

- Gutters are used to convey water from the roof to pipes to the storage tank or cistern.
- If a straight run of gutter exceeds 20 m, use an expansion joint.
- Keep the front of the gutter 15 mm lower than the back.
- Provide a minimum gutter slope of 1:200.
- Gutter should be a minimum of 26 gauge galvanized iron or 22 gauge Aluminium.
- Down spout should provide 6 square cm of opening for every 10 square m of roof area.
- The maximum run of gutter for one down pipe is 15 m.

b) Size of Rain Water Pipes for Roof drainage

The pipe diameter and average rainfall are two parameters that are considered in sizing of Rain water pipes to enable cater to roof surface area. This is given in Table 9.2. The table is to provide data for number of pipes of a particular data needed for various roof surface areas and average rate of rainfall in mm/hour.

Table 9.2: Sizing Rain Water pipes for Roof Surface area drainage

Diameter pipe(mm)	Average rate of rainfall (mm/hr)					
	50	**75**	**100**	**125**	**150**	**200**
	Roof area (m^2)					
50	13.4	8.9	6.6	5.3	4.4	3.3
65	24.1	16.0	12.0	9.6	8.0	6.0
75	40.8	27.0	20.4	16.3	13.6	10.2
100	85.4	57.0	42.7	34.2	28.5	21.3
125	-	-	80.5	64.3	53.5	40.0
150	-	-	-	-	83.6	62.7

Source: CPWD Manual

The storage system ensures water for continuous supply even during dry periods. The storage tank is designed on the basis of a mechanism which store water during excessive raining and thus facilitating the use of the stored water during dry period.

The storage tanks are designed based on requirements of water, average annual rainfall and size of catchment. Designing the storage tank capacity for dry period is calculated as follows:

(i)	Area of roof top	: 100 m^2
(ii)	Average rainfall	: 1000 mm
(iii)	Coefficient of evaporation, spillage and first flush diversion	: 0.80

- Tank capacity for 245 days Dry period = 100 sqm x 1 m x 0.80 = 80,000 ltrs.
- Drinking water, for family of 5 members = 10 lts / person x 5 members x 245 days = 12,250 ltrs.
- Add safety factor of 20 % = 1.20 x 12,250 = 14,700 litres.

Hence, a rectangular tank with a depth of 2.5 m, length = 2.5 m, breadth of 2.5 m

Harvested roof top rainwater can be used for domestic purposes. However, in water scarce areas, they can be used for drinking purposes also after proper treatment and disinfection.

9.2.2.2 Percolation of runoff into ground

Rainwater collected from roof catchment can also be recharged to aquifer through suitable structure such as Percolation pits, percolation trenches and recharge wells etc.

9.2.2.2.1 Percolation pits

This method is suitable where a permeable stratum is available at shallow depth. It is adopted for buildings having roof area up to 100 sqm. Recharge pit of any shape is constructed generally 1-2 m wide and 2-3 m deep. The pit is filled with boulders, gravel and sand for filtration of rain water. Water entering in to RWH structure should be silt free. Top layer of sand of filter should be cleaned periodically for better ingression of rain water in to the sub soil. Details are shown in Fig. 9.2.

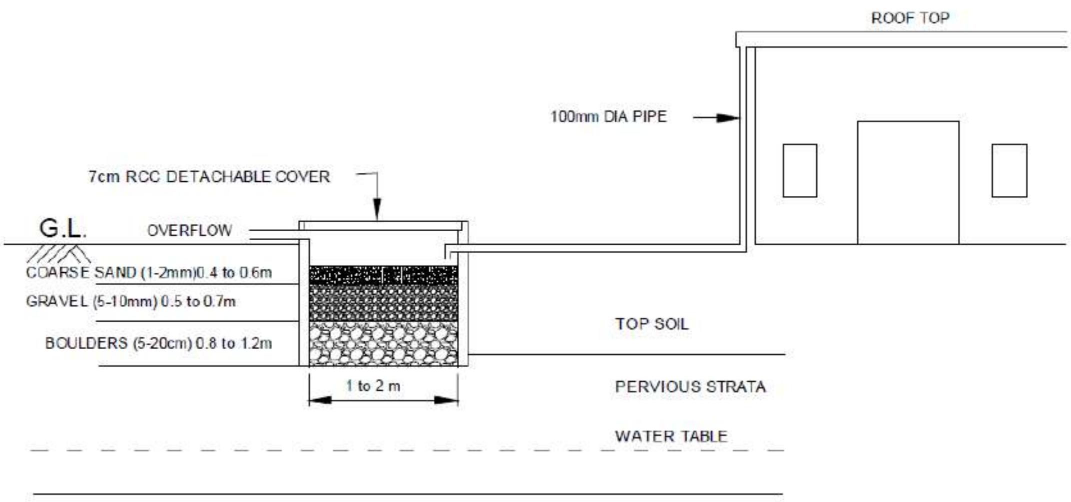


Figure 9.2: Percolation Pit

9.2.2.2.2 Percolation trenches

This method is used where permeable strata is available at shallow depth. It is suitable for buildings having roof top area between 200 & 300 sqm. In this method, trench of 0.5-1.0 m wide, 1-1.5 m deep and of adequate length depending upon roof top area and soil/subsoil characteristics should be constructed and filled with boulders, gravel and sand as shown in Fig. 9.3. Cleaning of filter media should be done periodically.

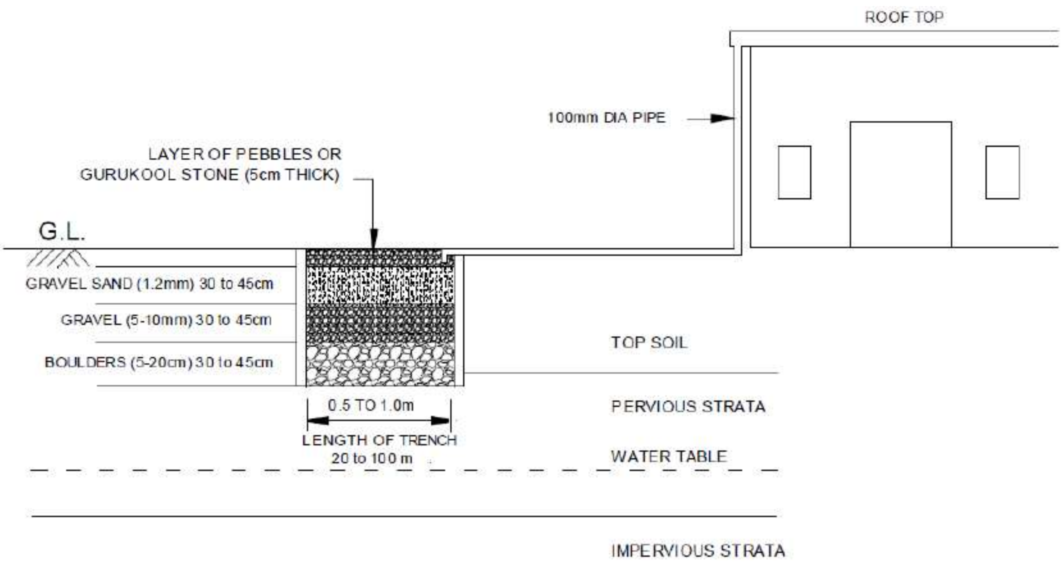


Figure 9.3: Percolation Trench

9.2.2.2.3 Recharge wells

In this method, a dry / unused dug well can be used as a recharge structure. It is suitable for buildings having a roof top area more than 100 sqm. Recharge water is guided through a pipe of 100 mm to the bottom of the well as shown in Fig. 9.4. Well cleaning and desilting is imperative before using it. Recharge water guided should be silt free, otherwise filter should be provided as shown in Fig. 9.4. Well should be cleaned periodically and chlorinated to control bacteriological contamination.

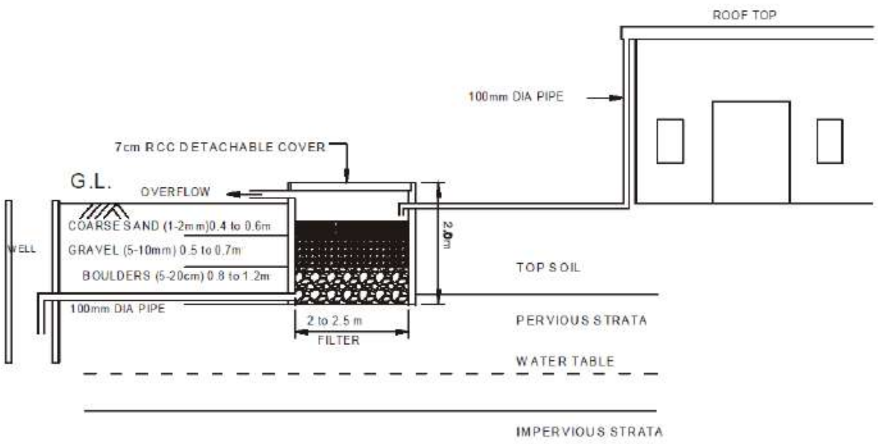


Figure 9.4: Recharge Wells

9.2.3 Storage of runoff in nearby percolation pond / water tank

Percolation tank is an artificially created surface water body, submerging in its reservoir a highly permeable land, so that accumulated runoff is made to percolate and recharge the ground water storage. Depending on requirement, it is possible to have more than one percolation tank in a catchment if sufficient surplus runoff is available and the site characteristics favour artificial recharge through such structures. In such situations, each tank of the group takes a share of runoff of the whole catchment above it, which can be classified in two categories i.e. free catchment and combined catchment as defined below:

(i) 'free catchment', which is the catchment area that only drains into the tank under consideration and
(ii) 'combined catchment', which is the area of the whole catchment above the tank.

The difference between the combined and free catchment gives the area of the catchment intercepted by the tanks located upstream of any tank. Each tank will receive the whole runoff from its free catchment, but from the remainder of its catchment it will receive only the balance runoff that remains after the upper tanks have been filled.

9.2.4 Percolation of storm water inside / outside the drains along its stretch

The existing drains in urban area are attractive infrastructure facilities for arresting surplus urban run-off which runs waste in monsoon period. Hydraulics of ground water recharge through surface drains should be evaluated for which input parameters such as recharge rate, free flow and detained flow can be used in the computation of total volume recharged. Recharge rates can be computed at different depths of flow in the drain. Accordingly volume of water recharged and rise in groundwater table can be assessed at, for different rainfall events. The recharge rate through surface drains increases with increase in depth of flow in the drain and volume of water recharged under detained flow conditions.

Rain water collection model through storm water drain in urban areas is depicted in Fig 9.5.

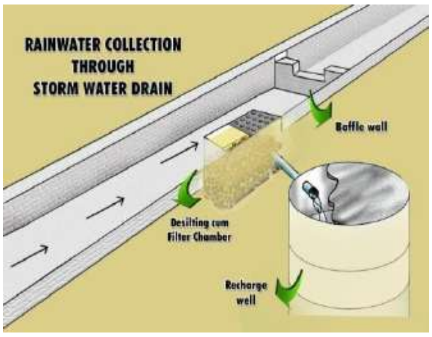


Figure 9.5: Rain Water Collection through storm water drains

9.2.5 Spreading water for recharge in low lying areas and park / gardens etc.

This technique is ideal for lands adjoining rivers or irrigation canals in which water levels remain deep even after monsoons and where sufficient non-committed surface water supplies are available. The schematics of a typical flooding system are shown in Fig 9.6. To ensure proper contact time and water spread, embankments are provided on two sides to guide the unutilized surface water to a return canal to carry the excess water to the stream or canal. Flooding method helps reduce the evaporation losses from the surface water system, is the least expensive of all artificial recharge methods available and has very low maintenance costs.

Figure 9.6: Water spreading in low lying area

Note: The detailed design criteria is elaborated in detail in 'Manual on Artificial Recharge of Ground Water', published by CGWB. The same can be accessed at www.cpheeo.gov.in.

9.2.6 Disposal to water body

After proper sedimentation, runoff from urban catchment should be disposed to the natural water bodies. Aerial extent of the water bodies and its capacity should be investigated to assess the requirement of the quantity of runoff from the catchment.

9.3 Precaution to be considered for harvesting of storm water

The storm water flow from a combined sewer shall never be taken up for recharge into the ground. Similarly, there would be situations wherein the sewage is getting mixed even in the drains meant exclusively for storm water and in such cases also the recharge of this sewage mixed with storm water shall be avoided unless or otherwise this is treated prior to a level fit for recharge. This shall be meticulously followed in order to avoid causing pollution.

9.4 Pollution of Storm Water and its Treatment

The storm water runoff from the urban areas during the first rains will come into contact with the pollution in the storm water drains, parking lots, etc. which have accumulated till that time. These can be night soil, urine, vegetable rejects, food rejects, dead insects, rats etc. and decaying papers etc. in the drains and chemical contaminants in the parking lot, etc. Once these are washed into the water courses, they pollute the same and may start water borne diseases like Cholera, Typhoid, Jaundice, etc in the waters. These organisms can also pollute the groundwater on river banks as well. The contaminants commonly found in storm water runoff and their likely sources are summarized in Table 9.3.

Table 9.3: Sources of Contaminants in Urban Storm Water Runoff

Contaminant	Contaminant Sources
Sediment and Floatables	Streets, lawns, driveways, roads, construction activities, atmospheric deposition, drainage channel erosion
Pesticides and Herbicides	Residential lawns and gardens, roadsides, utility right-of-ways, commercial and industrial landscaped areas, soil wash-off
Organic Materials	Residential lawns and gardens, commercial landscaping, animal wastes
Metals	Automobiles, bridges, atmospheric deposition, industrial areas, soil erosion, corroding metal surfaces, combustion processes

Contaminant	Contaminant Sources
Oil and Grease/ Hydrocarbons	Roads, driveways. parking lots. vehicle maintenance areas, gas stations. illicit dumping to storm drains
Bacteria and Viruses	Lawns, roads, leaky sanitary sewer lines, sanitary sewer cross-connections, animal waste, septic systems
Nitrogen and Phosphorus	Lawn fertilizers, atmospheric deposition. automobile exhaust, soil erosion, animal waste, detergents

9.5 Treatment methods for urban storm runoff

The onsite treatment methods of storm water are as below:

9.5.1 Sand Filters

Sand filters provide storm water treatment for first flush runoff. The runoff is filtered through a sand bed before being returned to a stream or channel. Sand filters are generally used in urban areas and are particularly useful for groundwater protection where infiltration into soils is not feasible. Alternative designs of sand filters use a top layer of peat or some form of grass cover through which runoff is passed before being strained through the sand layer. This combination of layers increases pollutant removal.

One of the main advantages of sand filters is their adaptability. They can be used on areas with thin soils, high evaporation rates, low soil infiltration rates, and limited space. Sand filters also have high removal rates for sediment and trace metals, and have a very low failure rate. Disadvantages associated with sand filters include the necessity for frequent maintenance to ensure proper operation, unattractive surfaces, and odour problems.

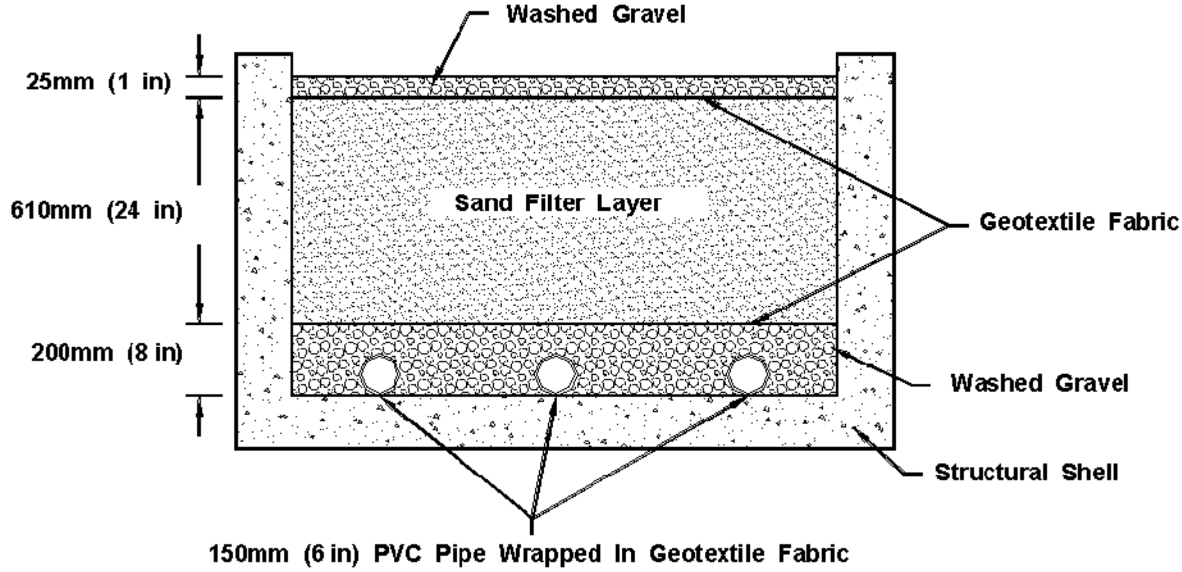


Figure 9.7: Cross-section schematic of sand filter compartment

Source: FHWA Manual

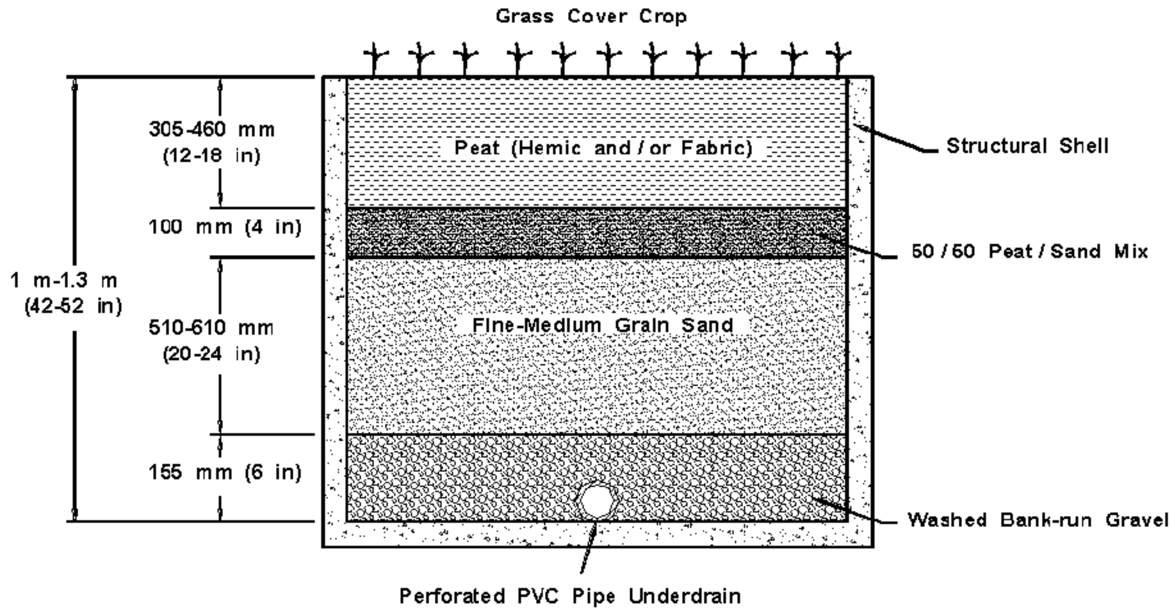


Figure 9.8:Cross-section schematic of peat-sand filter

9.5.2 Water Quality Inlets

Water quality inlets are pre-cast storm drain inlets that remove sediment, oil and grease, and large particulates from parking lot runoff before it reaches storm drainage systems or infiltration BMPs. They are commonly known as oil and grit separators. Water quality inlets typically serve highway storm drainage facilities adjacent to commercial sites where large amounts of vehicle wastes are generated, such as gas stations, vehicle repair facilities, and loading areas. They may be used to pre-treat runoff before it enters an underground filter system. The inlet is a three-stage underground retention system designed to settle out grit and absorbed hydrocarbons.

An oil and grit separator consists of three chambers as shown in Figure 9.9; a sediment trapping chamber, an oil separation chamber, and the final chamber attached to the outlet. The sediment trapping chamber is a permanent pool that settles out grit and sediment, and traps floating debris. An orifice protected by a trash rack, connects this chamber to the oil separation chamber. This chamber also maintains a permanent pool of water. An inverted elbow connects the separation chamber to the third chamber. Advantages of the water quality inlets lie in their compatibility with the storm drain network, easy access, capability to pre-treat runoff before it enters infiltration BMPs, and in the fact that they are unobtrusive. Disadvantages include their limited storm water and pollutant removal capabilities, the need for frequent cleaning (which cannot always be assured), the possible difficulties in disposing of accumulated sediments, and costs.

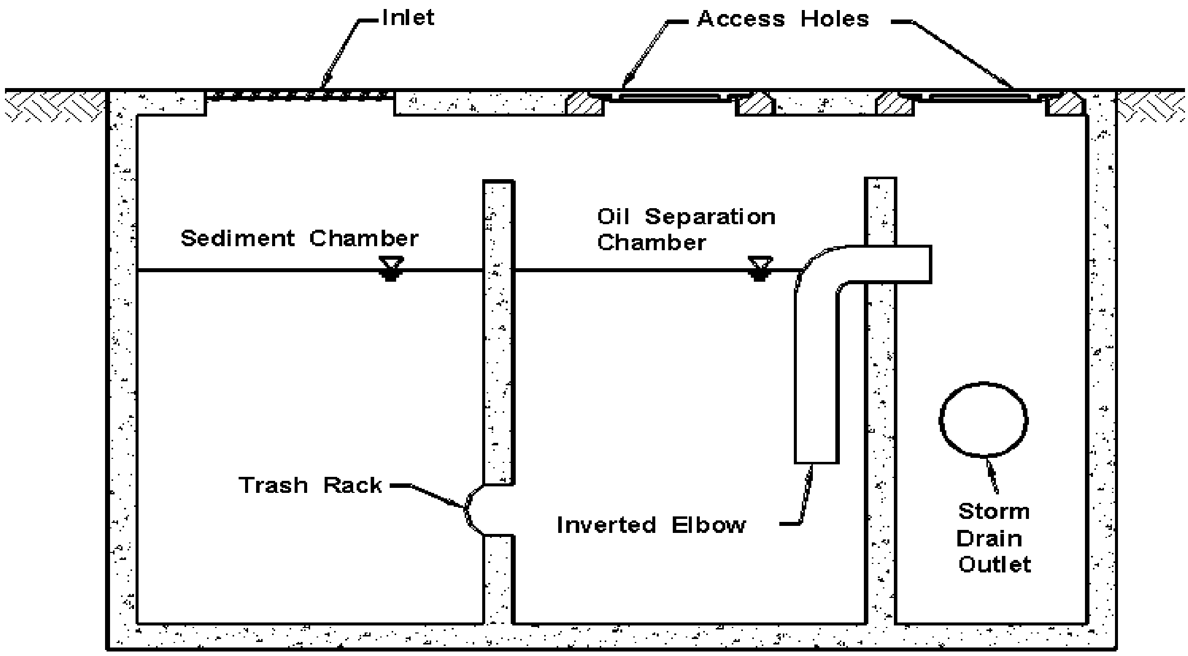


Figure 9.9: Water Quality Inlet

9.5.3 Filter Strips

Filter strips are similar in many respects to grassed swales, except that they are designed to only accept overland sheet flow. Runoff from an adjacent impervious area must be evenly distributed across the filter strips.

To work properly, a filter strip must be (1) equipped with some sort of level spreading device, (2) densely vegetated with a mix of erosion resistant plant species that effectively bind the soil, (3) graded to a uniform, even, and relatively low slope, and (4) be at least as long as the contributing runoff area.

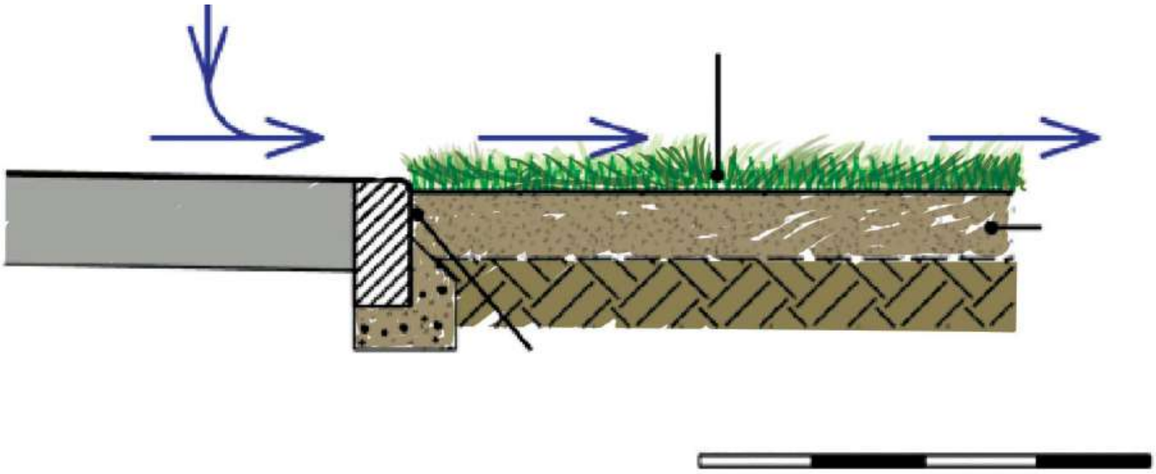


Figure 9.10: Cross section of Filter Strip

CHAPTER 10: INNOVATIVE STORM WATER MANAGEMENT PRACTICES

10.1 General

In today's urban centred growth, integration of innovative approaches for storm water management is getting prominence in city planning. In such approaches, development is allowed without significantly altering the pre-developed ground water recharge scenario. Storm water is now increasingly being valued as a resource to address water security in urban areas. To further strengthen water security, some developed countries use storm water for recharging ground water aquifer after important adequate treatment to pollution laden urban runoff. Further, some developed countries have taken it to next level, whereby, they are integrating smart practices such as Water Sensitive Urban Design (WSUD), Sustainable Urban Drainage System (SuDS), Low Impact Development (LID) and other Best Management Practices (BMP) in their urban planning to economize storm water management on one hand and water security on another hand. In this Chapter, a brief description of these innovative approaches is given to sensitize the users. The detailed design of each of these methods requires expertize and detailed analysis of existing ground situation for its integration in city infrastructure and is beyond the scope of this Manual. However, much country has come out with detailed guideline / Manuals for integrating above concepts in city urban planning.

10.2 Innovative Storm Water Management Practices

Many countries are coming out with innovative storm water practices, suiting to their socio economic and geographical condition. Under these practices, the maximum utilization of water resources is targeted with minimum investment while keeping the development in harmony with environment i.e. a huge shift from conventional storm water drainage system designs. Several models have been attempted across the world suiting to local conditions. The following 3 models are prominent and can be integrated in the storm water drainage planning and designing. A brief of these models is presented below.

10.2.1 Water Sensitive Urban Design (WSUD) (Australian Model)

A new approach termed 'Water Sensitive Urban Design' (WSUD) was developed in the late 1980s for urban planning and design. WSUD provides a broad framework which incorporates storm water related issues like water quality, water quantity and its conservation on one hand and integration of water security, waste water treatment & reuse, protection of water bodies and environmental & social objectives on other hand. In nutshell, the paradigm shift under WSUD is to see stormwater as a valuable resource in conjunction with water and treated waste water and not a mere traditional design for its conveyance and disposal. The various aspects considered under WSUD is presented in the figure below:

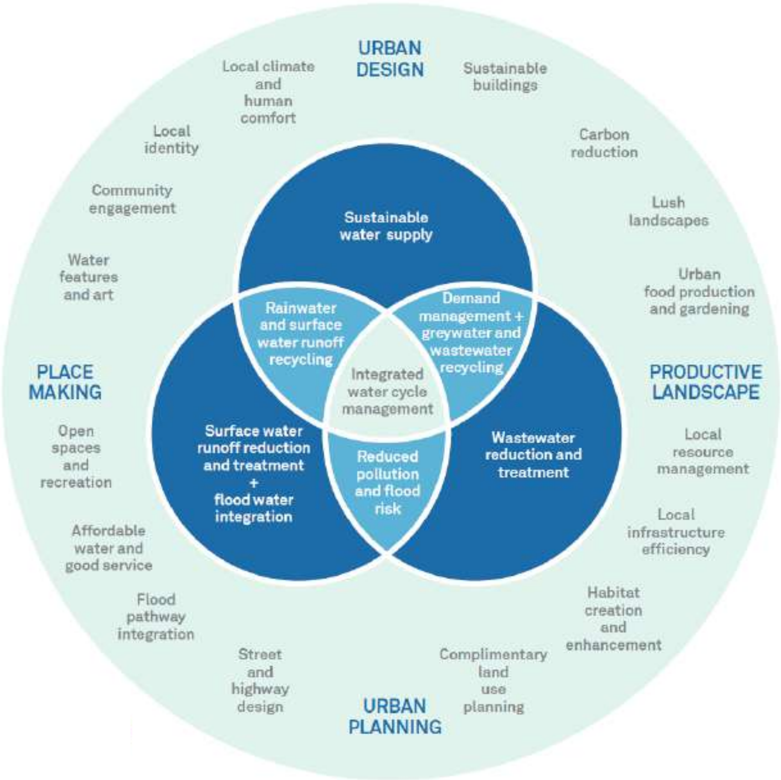


Figure 10.1: Aspects of Water Sensitive Urban Design

Source: Water Sensitive Urban Design in the UK –Ideas for built environment practitioners - a scoping study (CIRIA project RP976); www.susdrain.org

10.2.1.1 Objective of WSUD

Water Sensitive Urban Design for urban storm water seeks to address, inter alia, the following important aspects:

i. Minimizing runoff at or near its source, by directing runoff from impervious surface to pervious areas to reduce quantity and improve quality of runoff
ii. Preserve the existing topography and features of the natural drainage system including waterways and water bodies.
iii. Integrate public open space with storm water drainage corridors to maximise public access, passive recreation activities and visual amenity, while preserving essential waterway habitats and wildlife movement corridors.
iv. Preserve the natural water cycle including minimising changes to the natural frequency, duration, volume, velocity, and peak discharge of urban storm water runoff.
v. Protect surface water and groundwater quality.
vi. Minimise the capital and maintenance costs of stormwater infrastructure.

10.2.1.2 Integration of WSUD in Urban planning and Implementation

a. <u>City Urban Planning</u>

Following eight components can be integrated in city urban planning for on-ground implementation to achieve the objectives of Water Sensitive Urban Design.

1. Protect water quality

i. Storm water remains clean and retains its high value
ii. Implement best management practice on-site.
iii. Implement non-structural controls, including education and awareness programs.
iv. Install structural controls at source or near source.
v. Use in-system management measures.
vi. Undertake regular and timely maintenance of infrastructure and streetscapes.

2. Protect infrastructure from flooding and inundation

i. Storm water runoff from infrequent high intensity rainfall events is safely stored and conveyed
ii. Safe passage of excess runoff from large rainfall events towards watercourses and wetlands.
iii. Store and detain excess runoff from large rainfall events in parks and multiple use corridors.
iv. Safely convey excessive groundwater to the nearest watercourse.

3. Minimise runoff

i. Slow the migration of rainwater from the catchment and reduce peak flows
ii. Retain and infiltrate rainfall within property boundaries.
iii. Use rainfall on-site or as high in the catchment as possible.
iv. Maximise the amount of permeable surfaces in the catchment.
v. Use non-kerbed roads and carparks.
vi. Plant trees with large canopies over sealed surfaces such as roads and carparks.

4. Maximise local infiltration

- *Fewer water quality and flooding problems*
- Minimise impervious areas.

- Use vegetated swales.
- Use soak wells and minimise use of piped drainage systems.
- Create vegetated buffer and filter strips.
- Recharge the groundwater table for local bore water use.

5. **Make the most of nature's drainage**

- *Cost effective, safe and attractive alternatives to pipes and drains*
- Retain natural channels and incorporate into public open space.
- Retain and restore riparian vegetation to improve water quality through bio-filtration.
- Create riffles and pools to improve water quality and provide refuge for local flora and fauna.
- Protect valuable natural ecosystems.
- Minimise the use of artificial drainage systems.

6. **Minimise changes to the natural water balance**

- *Avoid summer algal blooms and midge problems and protect our groundwater resources*
- Retain seasonal wetlands and vegetation.
- Maintain the natural water balance of wetlands.
- No direct drainage to conservation category wetlands or their buffers, or to other conservation value
- Wetlands or their buffers, where appropriate.
- Recharge groundwater by storm water infiltration.

7. **Integrate storm water treatment into the landscape**

- Add value while minimising development costs
- Public open space systems incorporating natural drainage systems.
- Water sensitive urban design approach to road layout, lot layout and streetscape.
- Maximise environmental, cultural and recreational opportunities.

8. **Convert drains into natural streams**

- *Lower flow velocities, benefit from natural flood water storage and improve waterway ecology*
- Create stable streams, with a channel size suitable for 1 in 1 year design return period rainfall events, equivalent to a bankfull flow.
- Accommodate large and infrequent storm events within the floodplain.

- Create habitat diversity to support a healthy, ecologically functioning waterway.

b. Step by Step Implementation

Step 1: Understand the Site

Step 1 in the design process is about developing a broad overview of the subject site and identifying those issues that may assist or hamper the overall delivery of WSUD practices.

Step 2: Identify Objectives

The implementation of WSUD in a development seeks to achieve a range of outcomes relating to water quality, hydrology, conservation, biodiversity and amenity. Each of these outcomes can be met by ensuring development complies with the appropriate objectives and targets identified for the site.

The objectives should focus on:

- Water quality;
- Water quantity;
- Integrated water cycle management;
- Landscape and amenity;
- Biodiversity enhancement; and
- Social outcomes.

Step 3: Identify Suitable WSUD Measures

To assess whether a WSUD measure is appropriate requires an understanding of the requirements of the WSUD outcomes and the suitability of the particular measure to assist in achieving those outcomes. In developing a proposed WSUD strategy, it is often necessary to review this on an iterative basis, so that the characteristics of different WSUD measures can be appropriately integrated.

Step 4: Meet with Council and Relevant Authorities

In the majority of situations, it will be beneficial to the overall development process to meet with council officers to:

- Discuss the site of the proposed development, including opportunities and constraints of the site;
- Discuss the concept design of the proposed development;
- Establish objectives and targets for the proposed development;
- Discuss any likely council requirements, including any modelling expectations;
- Discuss land and asset ownership issues including future maintenance and operation; and

- Determine the necessary approvals including any State Government approvals.

Step 5: Conceptual Site Design

WSUD principles are most effective and economical when integrated into development design at the concept design stage. Each development type may vary significantly and present different WSUD opportunities. There are many ways to incorporate WSUD in development projects to meet the objectives and targets. The design strategies used in a project will depend upon:

- The location and geography of the site;
- Land use and activity (residential, commercial, industrial);
- Development or redevelopment scale;
- Water use and demand (garden irrigation, industrial needs, etc.);
- Water sources available, including rainfall, storm water and wastewater;
- On-site catchment area (roof and surface);
- Groundwater and soil type;
- Infrastructure (building and roads);
- Surrounding environment opportunities and constraints;
- Operation and maintenance (council or site owner);
- Urban landscape design (architectural and landscape); and
- Catchment water quantity and quality objectives and targets.

Step 6: Model Base Case (if required by approving authority)

At this stage, sufficient information would have been collected to allow modeling of both the existing site (i.e. pre-development) and the 'untreated' developed site that would form the 'base case' with which to compare future modeling of the WSUD systems proposed for the development (if required by the approving authority). In the majority of developments, water quality modeling should focus on total suspended solids, total nitrogen, total phosphorus and gross pollutants as the key pollutants of interest, in addition to the hydraulic outcomes. Faecal coliforms and organics should also be considered, depending on the measure being assessed.

Step 7: Locate WSUD Measures

When determining the optimal WSUD measures for a site, some consideration should be given to the site analysis and the opportunities available, and the 'natural' or obvious areas for WSUD measures (e.g. overland flow paths). The site analysis may provide information on whether a 'bottom of catchment' approach or a distributed approach to WSUD is optimal for the site.

Step 8: Model Treated Case (if required by approving authority)

Evaluation and assessment of alternative water management strategies are based on predictions made using forecasting tools. The emergence of new models and design methods to evaluate the use of roof water and storm water, and reuse of treated wastewater allow more reliable assessment of the multiple benefits of utilising these alternative sources.

Step 9: Objectives Check

At this stage, several iterations may be required to ensure that the majority of objectives set out in Step 2 are achieved. Note that it may not be possible for all objectives to be met and it may be that a degree of compromise is required in some areas to achieve an optimal outcome. Where necessary, if particular objectives are essential, then it may be appropriate to revisit the conceptual site design and/or the type of WSUD measures used.

Step 10: Finalise Measures

Once the final WSUD conceptual design has been developed, it will be necessary to confirm sizing and locations of measures prior to entering the detailed design process. Of key importance at this stage will be the identification of services and completed design elements (e.g. roads, open space areas, final lot layouts, hydraulic design) within which WSUD measures may need to be integrated.
A conceptual design should be developed that shows:

- The location of the WSUD measure(s) within the development;

- The proposed layout of the measure in its specific location (also showing key features such as roads and other services). The proposed layout should also provide detail of proposed access to the WSUD measure for maintenance and monitoring and, where relevant, any surrounding recreational infrastructure. This is to ensure that adequate consideration has been given to ongoing maintenance and that the functionality of open and other recreational spaces is not impeded. Designers may also use the Design Assessment Checklist in each chapter during the concept design to check that no key issues will arise later in the detailed design. At this stage, it will also be appropriate to document operation and maintenance plans, including all ongoing requirements of each of the measures.

10.2.2 Low Impact Development Design (USA Model)

LID is an innovative storm water management approach modeled after nature i.e. **manages rainfall runoff at the source using uniformly distributed decentralized micro-scale controls.** LID is “a storm water management and land development strategy that emphasizes conservation and the use of on-site natural features integrated with engineered, small-scale hydrologic controls to more closely reflect pre-development hydrologic functions”. This can be accomplished by creating site design features that direct runoff to vegetated areas containing permeable or amended soils, protect native vegetation and open space, and reduce the amount of hard surfaces and compaction of soil. Common LID planning practices include site design planning based on natural land contours and decreasing the impervious surface. These methods include the following:

- Reducing impervious surfaces
- Disconnecting impervious areas
- Conserving natural resources
- Using cluster/consolidated development
- Using xeriscaping and water conservation practices

The basic LID strategy for handling runoff is to reduce the volume and decentralize flows. This is usually best accomplished by creating a series of smaller retention or detention areas that allow localized filtration instead of carrying runoff to a remote collection area for treatment. The basic LID strategy is explained schematically in figure below:

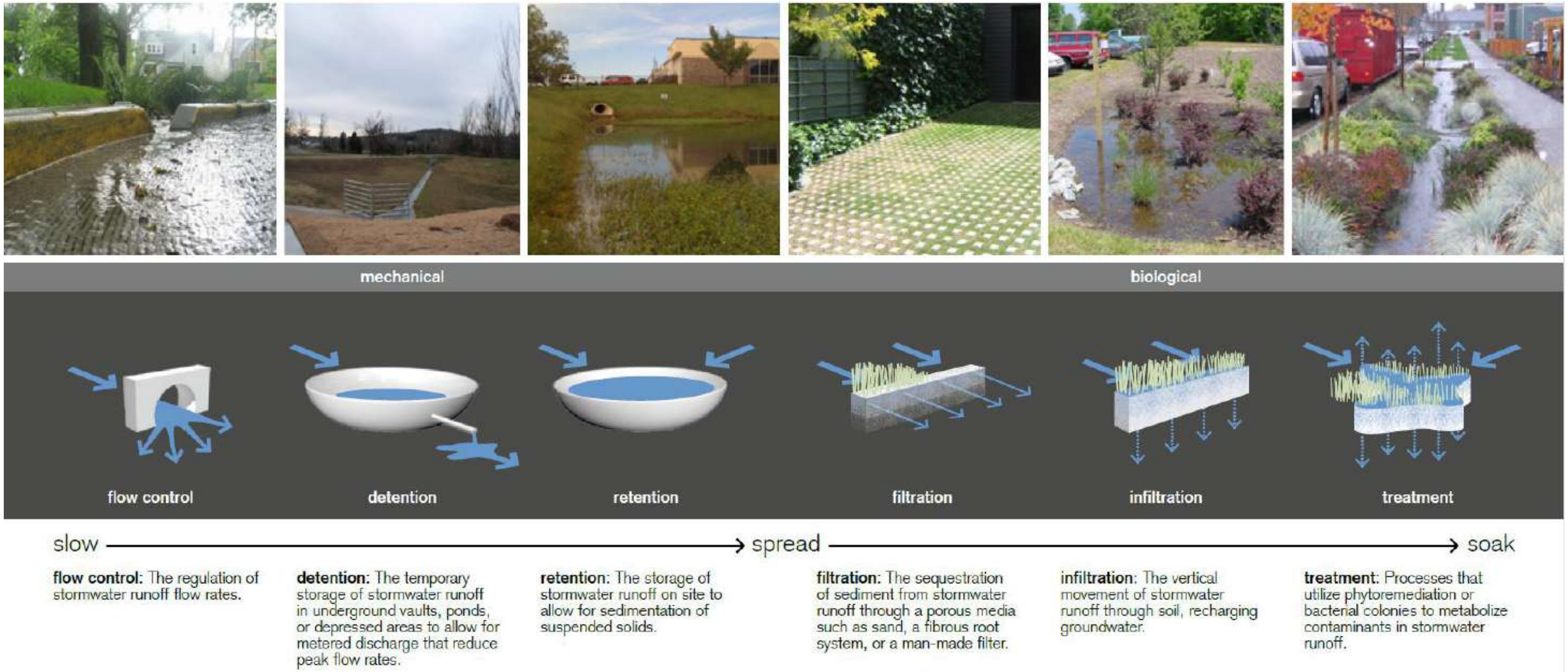


Figure 10.2: Basic LID strategy

Source: Low Impact Development, design manual for urban areas, University of Arkansas Community Design Center, Fayetteville, North Carolina, United States.

10.2.2.1 Objectives of Low Impact Development

Objectives of Low Impact Development (LID) are as follows:

- Protect water quality
- Reduce runoff
- Reduce impervious surfaces
- Encourage open space
- Protect significant vegetation
- Reduce land disturbance

10.2.2.2 Approach for Planning & Implementation of LID

Approaches for achieving objectives of LID can be broadly classified in following categories:

i. Site Planning
ii. Hydrologic Analysis
iii. Integrated Management Practices
iv. Erosion and Sediment Control
v. Public Outreach Program

The same is schematically represented in the figure below:

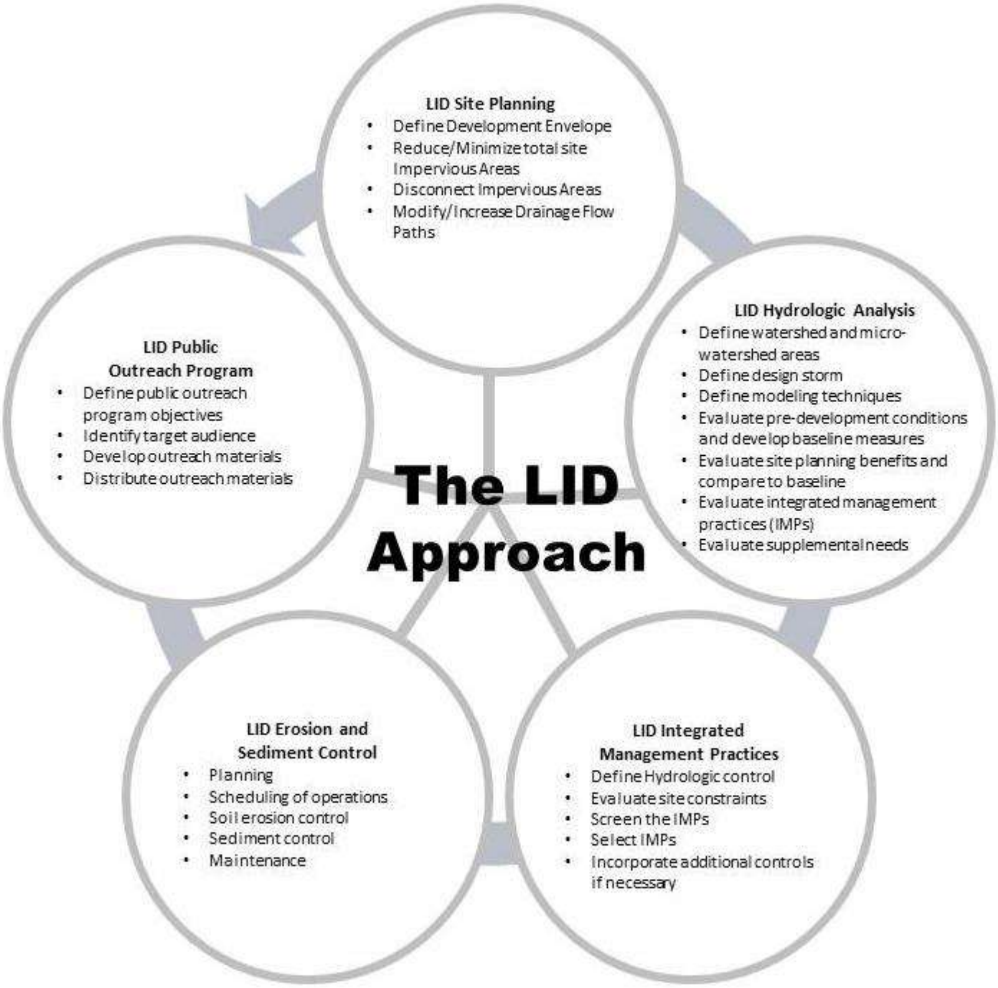


Fig 10.3: LID planning and implementation Approach

Source: Low-Impact Development Design Strategies, An Integrated Design Approach, Prepared by: Prince George's County, Maryland Department of Environmental Resources Programs and Planning Division, June 1999

To achieve above objectives, a step-by-step process for LID design development is described below:

STEP 1: DEFINE PROJECT OBJECTIVES AND GOALS

a. Identify the LID objectives for the entire project.
b. Determine the goals and feasibility for water quality, water quantity, peak runoff control, and on-site use of storm water.
c. Determine project character/aesthetic. Identify the baseline principles from which LID design decisions will be made by defining the LID technologies that support the concept and visual aesthetic.
d. Prioritize and rank basic objectives.

STEP 2: ANALYSIS AND SITE EVALUATION

A site evaluation will facilitate LID design development by providing infrastructural, contextual, cultural, and community clues that will assist in the development of a LID program.

a. Conduct a detailed investigation of the site through collected materials such as drainage maps, utilities information, soils maps, land use plans, and aerial photographs.
b. Perform an on-site evaluation highlighting opportunities and constraints, such as pollutant hot spots, potential disconnects from Combined Sewer Overflows, slopes etc. Make note of potential LID practices and areas where water quality and quantity controls could be installed.

STEP 3: CREATE OVERLAY

a. Classify the land use on the project site.
b. Review the proposed architectural plan to identify buildings and structures, open or vegetated space, parking lots, parking lot islands, side yards, vegetated strips adjacent to sidewalks, and buffer areas.
c. Create an overlay that identifies opportunities for LID devices.

STEP 4: DEVELOP LID CONTROL STRATEGIES

a. Develop a list of LID control strategies that potentially fulfill the objectives. Determine the appropriate number of LID controls needed. Identify specific LID technologies for the project site and determine how to integrate them, keeping in mind the optimum location, to meet their design objectives.
b. Specify LID technologies for each land use component.

STEP 5: DESIGN LID MASTER PLAN

a. Sketch a design concept that distributes the LID devices uniformly around the project site. Keep in mind that some LID technologies can be used to capture storm water from adjacent impervious areas.
b. Develop a master plan that identifies all key control issues (water quality, water quantity, water conservation) and implementation areas.
c. Finalize the plan.

STEP 6: DEVELOP SCHEDULE, FUNDING, CONSTRUCTION, AND IMPLEMENTATION PLANS

The development process is not a linear or static process but one that is dynamic and adaptable.

STEP 7: EVALUATE SUCCESS OR MODIFY DESIGN

Developing a storm water management program using LID principles and practices is a dynamic process. Evaluate the design to see if it meets project storm water management objectives.

10.2.3 Sustainable Drainage System (SuDS) (France Model)

Sustainable drainage systems aim towards maintaining or restoring a more natural hydrological regime, such that the impact of urbanisation on downstream flooding and water quality is minimised. Originally, SuDS were introduced primarily as single purpose facilities however this has now evolved into more integrated systems which serve a variety of purposes, including habitat and amenity enhancement.

SuDS involve a change in our way of managing urban run-off from solely looking at volume control to an integrated multi-disciplinary approach which addresses water quality, water quantity, amenity and habitat.

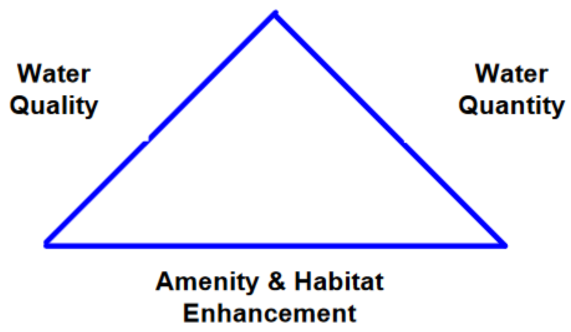
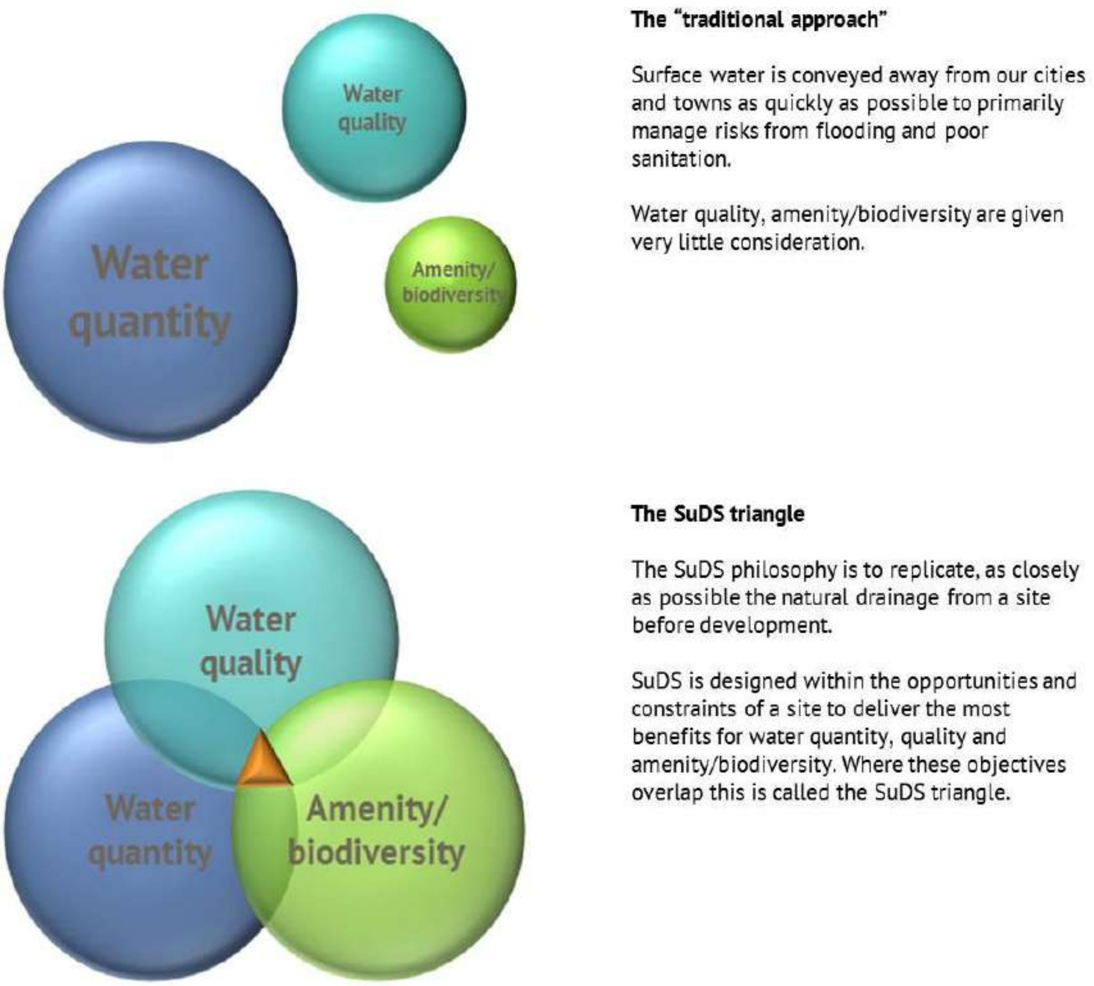


Fig 10.4: Sustainable Urban Drainage Concept

The difference between traditional approach and SuDS approach is explained in the figure below:



Source: www.susdrain.org

10.2.3.1 Objective of SuDS

Sustainable drainage systems (SuDS) are a natural approach to manage drainage in and around properties and other developments. SuDS work by slowing and holding back the water that runs off from a site, allowing natural processes to break down pollutants. Following two main objective of SuDS are:

1. Minimise the impacts of urban runoff by capturing runoff as close to source as possible and then releasing it slowly
2. Reduces pollutants in the surface water by settling out suspended solids

10.2.3.2 Steps for Design & Implementation of SuDS

SuDS Design can be primarily categorized in three types for implantation purposes:

i. **Source control** measures deal with run-off at, or close to, the surface where rainfall lands.

ii. **Site control** measures manage the surface water run-off from larger areas, such as part of a housing estate, major roads or business parks. The run-off from larger areas can be channelled to a site control measure using swales (shallow drainage channels) or filter drains.

iii. **Regional control** measures downstream of source and site controls deal with the gathered run-off from a large area. These systems use the same principles as smaller scale SuDS, but can cope with larger volumes of water. Rainwater that passes through small SuDS can feed into larger SuDS which deal with the gathered run-off from a wide area. It is best to connect the flows between SuDS components with swales, filter drains or ditches and avoid the use of pipes.

The SuDS planning process is schematically explained in Fig below:

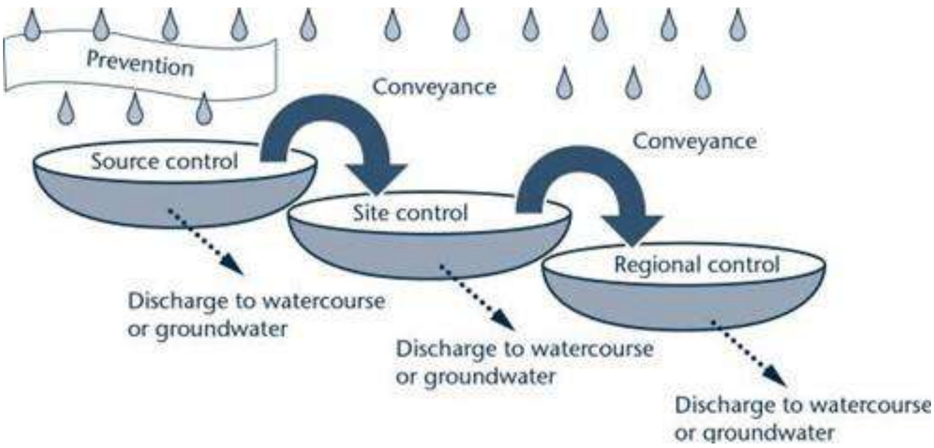


Fig 10.5: SuDS planning process

Source: www.susdrain.org

i. **Source Controls** manage storm water runoff as close to its source as possible, typically on site. Typical SuDS options include: green roofs, rainwater harvesting, permeable pavements and soakaways.
ii. **Local Controls** manage storm water runoff in the local area, typically within the road reserves. Typical SuDS options include: bio-retention areas, filter strips, infiltration trenches, sand filters and swales.
iii. **Regional Controls** manage the combined storm water runoff from several developments. Typical SuDS options include: constructed wetlands, detention ponds and retention ponds.

As the treatment train progresses, the number of interventions decrease, but their individual size increases. For example the source controls could be each house having a rainwater tank, the local control may be 5 houses ‘share’ a wet swale, and the regional control may be that 50 houses ‘share’ a wetland. The treatment train is shown in the Figure 10.6:

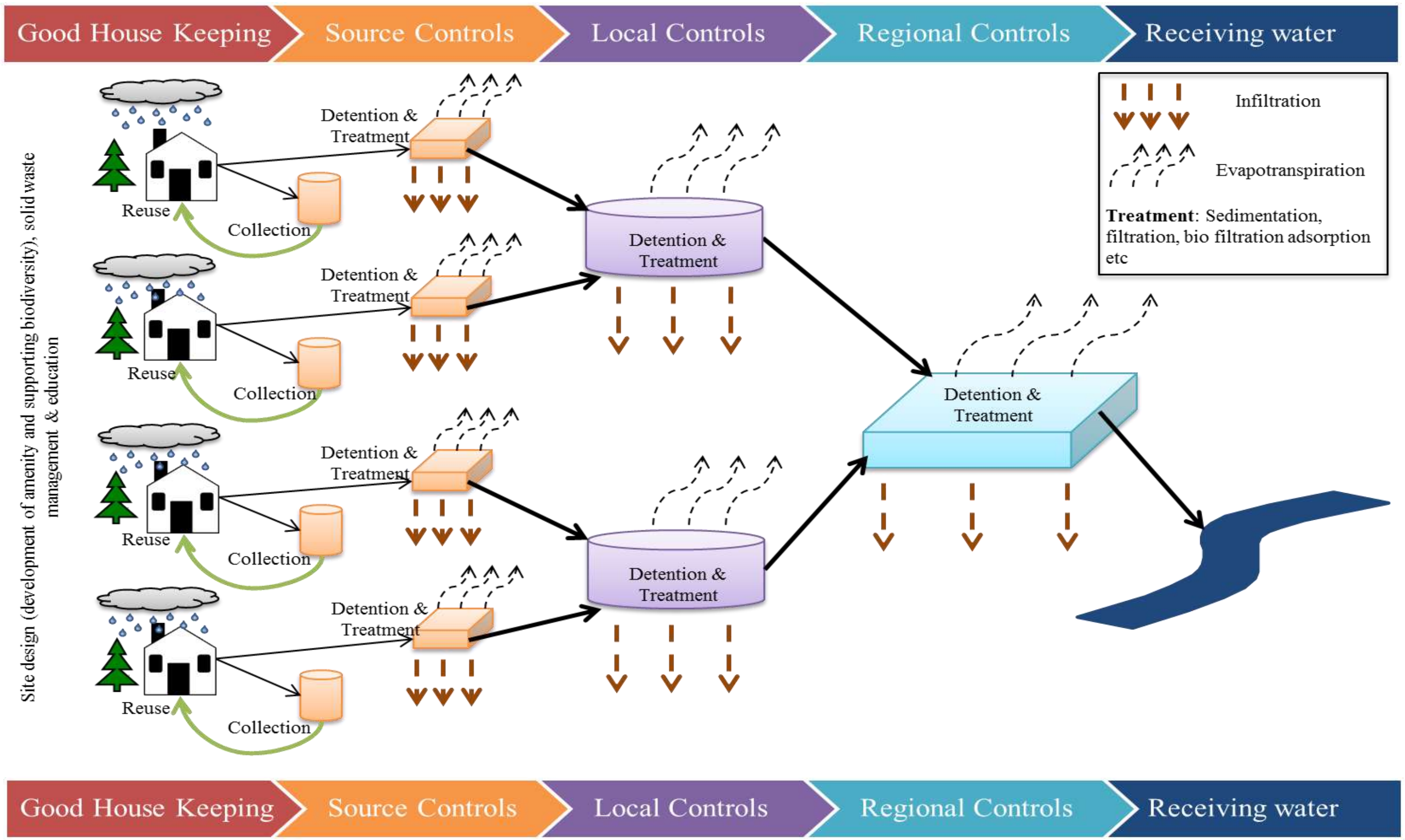


Figure 10.6: SuDS Treatment Train

Source: http://www.uwm.uct.ac.za/uwm/suds/principles

The successful achievement of sustainable urban drainage does not solely rely on the use of engineered techniques to control and treat runoff. 'Good housekeeping' measures, such as safe storage and handling of oils and chemicals, street sweeping and control of sediment run-off from construction sites are an essential component of SuDS. Public awareness is also an important factor in ensuring the successful implementation of sustainable drainage practices.

10.2.3.3 Benefits of SuDS

The benefits of SuDS are:

- Preventing water pollution
- Slowing down surface water run-off and reducing the risk of flooding
- Reducing the risk of urban flooding during heavy rain
- Recharging groundwater to help prevent drought
- Providing valuable habitats for wildlife in urban areas
- Creating green spaces for people in urban areas.

SuDS are very flexible and there are a number of ways that they can applied to provide great drainage that are both value for money and inspirational.

10.2.4 Decentralized Innovative intervention in storm water drainage designs

Storm water runoff control at decentralized level is of immense use and not only meets the local requirement of water for domestic uses but also can recharge ground water and minimize investment in storm water drainage infrastructure. In addition, it can contribute immensely to prevent frequent flooding in low lying areas. Decentralized innovative interventions that may be integrated in design / incentivize to realize above mentioned benefits are given in Table10.1:

Table 10.1 Decentralized Innovative intervention in storm water drainage designs

Type of development	Design element	Description and objectives	Example
	Rainwater harvesting	Element to collect rainwater from roofs and use for non-potable water uses	

Type of development	Design element	Description and objectives	Example
Residential	Permeable pavement	Permeable surface that rains through voids between solid parts of the pavement to infiltrate rainwater from pavements residential areas	
	Rain garden	Element vegetated to infiltrate rainwater from roofs	
	Infiltration trench	Stone-filled trench to infiltrate rainwater from roofs and pavements residential areas	
	Soakaway	Sub-surface structure to infiltrate rainwater from roofs	
Sidewalk	Rain garden	Element vegetated to infiltrate rainwater from sidewalk areas	
	Rill	Open vegetated channel to transport rainwater to infiltration areas	

Type of development		Design element	Description and objectives	Example
Street		Channel	Channel to transport rainwater to infiltration areas	
Street		Permeable pavement	Permeable surface that drains through voids between solid parts of the pavement to infiltrate rainwater from sidewalk areas	
Street	Road	Infiltration trench	Stone-filled trench to infiltrate and store rainwater from roads	
Street	Road	Pervious Pavement	Pervious surface that drains through voids between solid parts of the pavement to infiltrate rainwater	
Parking		Modular pavement	Modular surface to infiltrate rainwater from the parking surface	
Parking		Infiltration trench	Stone-filled trench to infiltrate rainwater from parking surface	

Type of development	Design element	Description and objectives	Example
	Bioretention	Depression backfilled with a soil mixture with vegetation to improve water quality from the parking surface	
Open space, flood plain, green infrastructure and infiltration area	Modular pavement	Modular surface to infiltrate rainwater	
	Natural pavement	Natural surface to infiltrate rainwater	
	Swale	Vegetated area to transport and infiltrate rainwater	
	Filter drain	Gravel trench to drain rainwater	
	Infiltration basin	Depression with vegetation area to infiltrate rainwater	

Type of development		**Design element**	**Description and objectives**	**Example**
		Wetland	Retention pond with aquatic vegetation to treat rainwater	
		Retention pond	Artificial pond to store water and release it slowly	
		Bioretention	Depression backfilled with a soil mixture with vegetation to retain rainwater	
Other spaces	Spaces between infrastructures	Infiltration basin	Depression with vegetation to infiltrate rainwater	
	Slope areas	Natural retention	Slope area with vegetation to infiltrate rainwater	

Above decentralized options are given to sensitize the users about their importance in local planning in the city and at individual household level and community level. However, detail design of each of these decentralized options to be carried out under guidance of experts and also based on the type designs available in typical Manuals like CGWB and CPWD etc.

CHAPTER 11: CONSTRUCTION OF STORM WATER CONDUITS / DRAINS

11.1 General

This chapter describes the method of construction of storm water drains and conduits such as laying and jointing of storm water conduits and construction of storm water drains, types of construction materials. Construction of manhole and other appurtenant structures etc has also been described.

11.2 Implementation of project

Before the implementation of any project, following prerequisites are mandatory:

i. Administrative approval
ii. Expenditure sanction
iii. Technical sanction
iv. Availability of funds

No work should normally be commenced or any liability thereon incurred until an administrative approval has been obtained, a properly prepared detailed estimate has been technically sanctioned and where necessary expenditure sanction has been accorded and allotment of funds made.

The execution of a project/work has two stages, viz. the '**Pre-construction stage**' and the '**Construction stage**'. The following activities are involved in these stages:

11.2.1 Pre-construction stage

Following prerequisites are mandatory before the construction work is taken up for the execution of the project:

1. Requisition from the client.
2. Preparation of site/soil data, and assessment of feasibility of services such as water supply, electricity, drainage and sewerage etc.
3. Approval of the preliminary plans by the client.
4. Preparation of preliminary estimate.
5. Approval of the preliminary estimate by the client.
6. Preparation and submission of the plans to the Local Bodies for their approval.
7. Approval of plans by the Local Bodies.
8. Preparation of preliminary structural sizes.
9. Preparation of structural drawings.
10. Preparation of detailed working drawings.

11. Preparation of detailed estimates for laying of storm water drains / conduits and all services (civil, electrical and mechanical).
12. Preparation of NIT and call of pre-qualification applications, wherever applicable.
13. Selection of contractors from the pre-qualification applications wherever applicable.
14. Call of tenders and pre-bid conference wherever applicable.
15. Receipt/Opening of tenders.
16. Decision on tender and award of work.

11.2.2 Construction stage

1. Execution of work and contract management.
2. Completion of work.
3. Testing and commissioning.
4. Completion certificate from Local Body including fire clearance.
5. Handing over to client.
6. Settlement of accounts

11.3 Construction of Storm water conduits

Steps involved in construction of storm water flexible and rigid pipe are as follows:

1. Removal of pavement and disposal of excavated materials from the ground.
2. Trench excavation.
3. Sheeting and bracing of the sides of the trenches wherever necessary to support the sides against caving.
4. Dewatering the trenches where necessary.
5. Protection of underground Services
6. Bedding, Laying and Jointing of Conduits
7. Backfilling of trenches
8. Removal of sheeting or bracing

11.3.1 Removal of pavement

The removal of pavement is often necessary as first step in conduit construction. It may be done by hammer and chisel or mechanically with pneumatic hammer fitted with various cutting tools. Excavated material should be safely disposed.

11.3.2 Trench Preparation

11.3.2.1 Dimensions

The width of a conduit trench depends on the soil condition, type of side protection and the working space required at the bottom of trench for smooth installations. Increase in width over required minimum would unduly increase the load on pipe and cost of road restoration. Considering all above factors, the minimum trench width is specified as per Table 11.1.

11.3.2.2 Excavation

Excavation of conduit trenches shall be in straight lines as much as possible and to the correct depths and gradients as specified in drawings. However, because of inherent flexible property, these pipes can also be laid at very wide and smooth curvatures without transitional manholes. Instead of conventional manholes, the specified fittings such as tees and bends, etc, can be used at transitions.

Table 11.1: Minimum Trench Widths

Sl No.	Pipe Diameter (mm)	Trench Width (m)
(1)	**(2)**	**(3)**
i)	75 to 200	0.6
ii)	250	0.7
iii)	300	0.8
iv)	400	0.9
v)	600	1.2
vi)	800	1.3
vii)	900	1.6
viii)	1000	1.8
ix)	1200	2.0

Excavated spoils shall not be deposited in the near proximity to prevent the collapse of side of the trenches. The sides of the trench shall, however, be supported by shoring (where necessary) to ensure proper and speedy excavations and concurrently ensuring necessary protections to contiguous structures. In the event, the presence of ground water is likely to cause instability in soil conditions, a well point system may be adopted for lowering of ground water table below the requisite trench bed level. If excavation is made deeper than necessary the same shall be filled and compacted.

11.3.2.3 Shoring/Mild steel sheet piling

The protective shoring works shall be strong enough to prevent caving in of trench walls or subsidence of contiguous areas adjacent to trench. For wider and deeper trenches, a system of wall plates (wales) and struts of heavy timber section is commonly used as per the requisite structural design. In non-cohesive soils with high ground water table, continuous interlocking mild steel sheet piling may be necessary to prevent excessive soil movements due to ground water percolation. Such sheet piling shall extend 1.5 m below the trench bottom unless the lower soil strata are adequately cohesive.

11.3.2.4 Underground services

The underground public and private utility services exposed due to the excavation shall be effectively supported under the guidance of the owners of such services.

11.3.2.5 Dewatering

Conduit installation trenches shall be adequately dewatered for the placement of pipe at proper gradient till the pipe is integrated through socket and spigot joint/coupler assembly with the already laid segment.

11.3.2.6 Floatation of Flexible pipe

Precautions are to be taken to arrest floating of installed conduit segments against buoyant forces in case of sudden accumulation of water in the trench. The diameter wise minimum cover necessary to counteract the buoyant forces is given in Table 11.2. For exceptional cases of higher level of ground water, additional anchoring at equal intervals would be necessary.

Table 11.2: Required minimum cover to prevent floatation

Sl No.	Nominal Diameter mm	Minimal Cover mm
(1)	(2)	(3)
i)	75	65
ii)	100	77
iii)	150	102
iv)	200	127
v)	250	178
vi)	300	368
vii)	400	505
viii)	600	711
ix)	900	1067

x)	1050	1219
xi)	1200	1372
NOTE- Computation is based on the pipes being completely empty, water table at the ground surface, solid density of 2083 kg/m^3 and a soil friction angle appropriate for most sand/ gravel mixture. The average of the inside and outside diameters was used to determine solid and water displacement.		

11.3.3 Pipe Bedding for rigid pipes

Where storm water conduits / drains has to be laid in a soft underground strata or in a reclaimed land, the trench shall be excavated deeper than what is ordinarily required. The trench bottom shall be stabilized by the addition of coarse gravel or rock. In case of very bad soil the trench bottom shall be filled in with cement concrete of appropriate grade. In the areas subject to subsidence, the pipe should be laid on suitable supports or concrete cradle supported on piles. In the case of cast-in-situ an RCC section with both transverse and longitudinal steel reinforcement shall be provided when intermittent variations in soil bearing capacity are encountered. In case of long stretches of very soft trench bottom, soil stabilization shall be done either by rubble, concrete or wooden crib.

11.3.3.1 Type of Bedding

The type of bedding (granular, concrete cradle, full concrete encasement etc.) would depend on the soil strata and depth at which pipe is laid. The load due to backfill, superimposed load (live load) and the three-edge-bearing strength of pipe (IS: 458) are the governing criteria for selection of appropriate bedding factors.
* Factor of safety = 1.5
The type of bedding to be used depends on the bedding factor and the matrix of type of bedding for different diameters and different depths has been tabulated in Table11.3 and Table 11.4.

Table 11.3: Type of bedding for storm water conduits

Bedding Factor	**Type of Bedding**
Up to 1.9	Class B Granular (GRB)
1.9 - 2.8	Class Ab: Plain Concrete Cradle(PCCB)
2.8 - 3.4	Class Ac : Reinforced Concrete cradle (RCCB) with 0.4 % Reinforcement
> 3.4	Class Ad : Reinforced concrete arch with 1.0% reinforcement

Table 11.4: Selection of bedding for different depths and different diameters

Diameter	Bedding type for cover depth in m				Diameter	Bedding type for cover depth in m			
mm	up to 2.5	2.5-3.5	3.5-5.0	5.0-6.0	mm	up to 2.5	2.5-3.5	3.5-5.0	5.0-6.0
400	Ab	Ab	Ab	Ac	1400	B	Ab	Ab	Ab
500	Ab	Ab	Ab	Ab	1500	B	Ab	Ab	Ab
600	B	Ab	Ab	Ab	1600	B	Ab	Ab	Ab
700	B	Ab	Ab	Ab	1800	B	Ab	Ab	Ab
750	B	Ab	Ab	Ab	2000	B	Ab	Ab	Ab
800	B	Ab	Ab	Ab	2200	B	Ab	Ab	Ac
900	B	Ab	Ab	Ab	2400	B	Ab	Ab	Ac
1000	B	Ab	Ab	Ab	2600	B	Ab	Ab	Ac
1200	B	Ab	Ab	Ab	2800	B	Ab	Ab	Ac

11.3.3.2 Classes of Bedding for Trench Conditions

Four classes, A, B, C and D, of bedding used most often for pipes in trenches are illustrated in Figure 11.1. Class A bedding may be either concrete cradle or concrete arch. Class B is bedding having a shaped bottom or compacted granular bedding with a carefully compacted backfill. Class C is an ordinary bedding having a shaped bottom or compacted granular bedding but with a lightly compacted backfill. Class D is one with flat bottom trench with no care being taken to secure compaction of backfill at the sides and immediately over the pipe and hence is not recommended.

Class B or C bedding with compacted granular bedding is generally recommended. Shaped bottom is impracticable and costly and hence is not recommended.

The pipe bedding materials must remain firm and not permit displacement of pipes.
The material has to be uniformly graded or well graded. Uniformly graded materials include pea gravel or one-size materials with a low percentage of over and undersized particles.

Well-graded materials containing several sizes of particles in stated proportions, ranging from a maximum to minimum size coarse sand, pea gravel, crushed gravel, crushed screenings, can be used for pipe bedding.

Fine materials or screenings are not satisfactory for stabilizing trench bottoms and are difficult to compact in a uniform manner to provide proper pipe bedding.

Well-graded material is most effective for stabilizing trench bottom and has a lesser tendency to flow than uniformly graded materials. However, uniformly graded material is easier to place and compact above pipes.

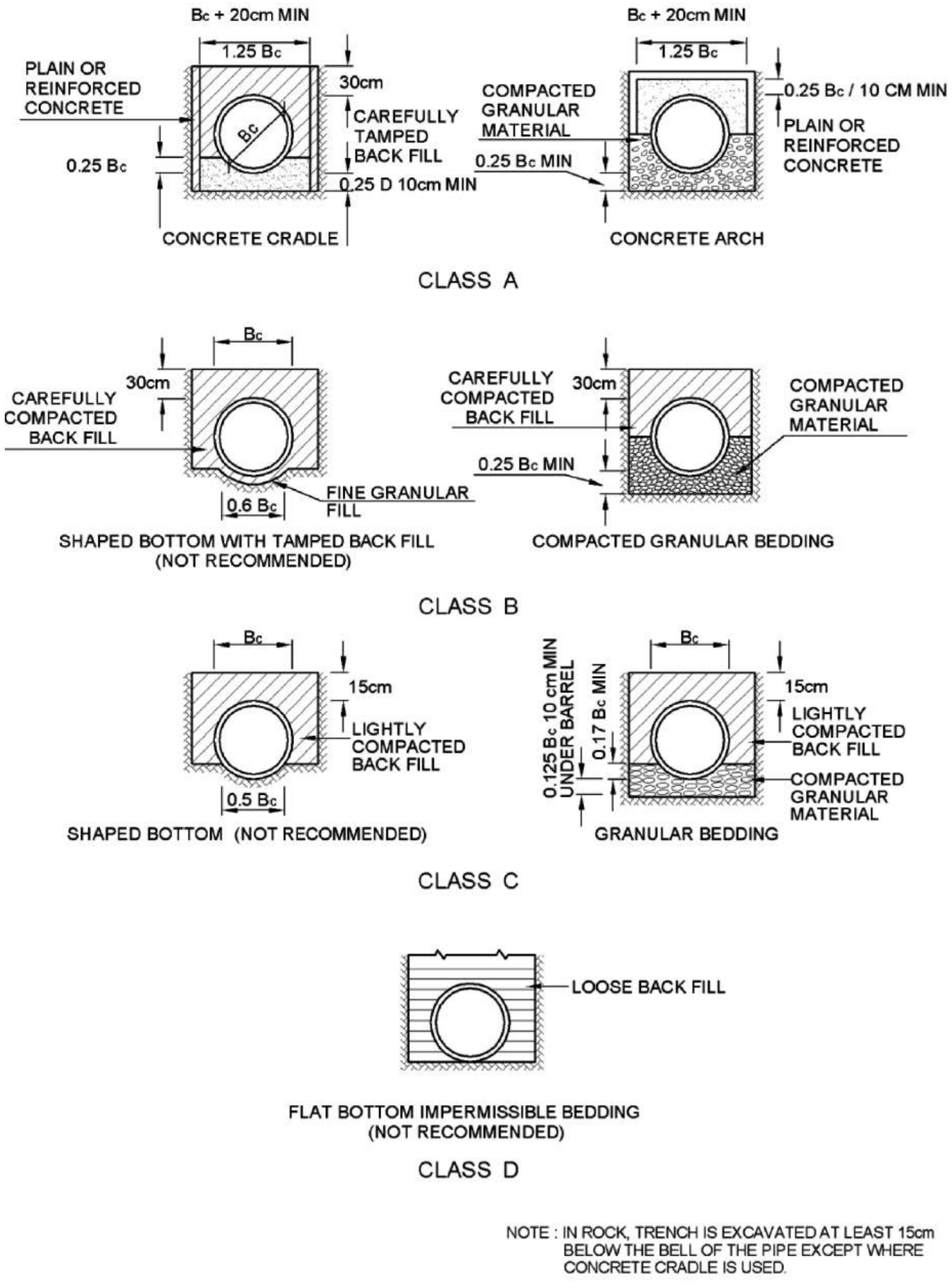


Figure 11.1: Classes of bedding

11.3.4 Bedding of Flexible pipe

a) Normally, even for the maximum combined loading (wheel load + backfill), any form of cement concrete structural bedding would not be necessary.
b) For maintenance of conduit slopes the initial backfill envelop with sand or gravel (as computed through structural design of buried flexible conduit) over a single BFS would be sufficient.
c) In the event, anchorage becomes imperative the transverse concrete anchorage blocks spaced at suitable interval shall also act as chairs for defining and maintaining the conduit slopes.

11.3.5 Laying of Rigid Storm water conduit

In laying pipe, the centre of each manhole shall be marked by a peg. Two wooden posts 100 mm x 100 mm and 1800 mm high shall be fixed on either side at nearly equal distance from the peg or sufficiently clear of all intended excavation. The sight rail when fixed on these posts shall cross the centre of manhole. The sight rails made from 250 mm wide x 40 mm thick wooden planks and screwed with the top edge against the level marks shall be fixed at distances more than 30 m apart along the pipe alignment. The centre line of the pipe shall be marked on the sight rail. These vertical posts and the sight rails shall be perfectly square and planed smooth on all sides and edges. The sight rails shall be painted half white and half black alternately on both the sides and the tee heads and cross pieces of the boning rods shall be painted black. When the conduits converging to a manhole come in at various levels, there shall be a rail fixed for every different level.

The boning rods with cross section 75 mm x 50 mm of various lengths shall be prepared from wood. Each length shall be a certain number of meters and shall have a fixed tee head and fixed intermediate cross pieces, each about 300 mm long. The top edge of the cross pieces shall be fixed at a distance below the top edge equal to, the outside dia. of the pipe, the thickness of the concrete bedding or the bottom of excavation, as the case may be. The boning staff shall be marked on both sides to indicate its full length.

The posts and the sight rails shall in no case be removed until the trench is excavated, the pipes are laid, jointed and the filling is started.

When large pipe lines are to be laid or where sloped trench walls result in top-of-trench widths too great for practical use of sight rails or where soils are unstable, stakes set in the trench bottom itself on the pipe line, as rough grade for the pipe is completed, would serve the purpose.

11.3.6 Laying and Jointing of Flexible pipe

11.3.6.1 Laying

11.3.6.1.1For shallow trenches

Place the pipe manually on the initial backfill envelop directly.

11.3.6.1.2 Deep trenches with shoring/mild steel sheet Piling

a) Make the trench reasonably free from ground water and other liquids.
b) Place the pipe on the top level cross-struts of the timber shoring/mild steel sheet piling frame work.
c) Dismantle one/two cross struts and lower the pipe to the immediate lower layer of the cross struts and re-fixes the struts immediately.
d) In the same manner, reach up to the initial back filling and place the pipe at proper slope.
e) Ensure anchorage, if any, after laying.

11.3.6.2 Jointing

Various methods for jointing such as regular coupler made by online process, spigot and sockets are used. The moulded socket will have a suitable internal surface with profiles ribs for insertion of the next pipe into it. The socket end of the pipe to be inserted will have corrugated outer layer. On first valley segment of corrugated pipe (destined to be pushed into the coupler) one elastomeric rubber ring needs to be placed which is pushed into the coupler socket. This provides sufficient gripping lock and leak proof joint. Similar system is also used for fabricated accessories or moulded fittings required such as tee, bends, elbows, reducer end caps for the purpose of installation of the system related to drainage/sewerage. For quality connections following steps are to be ensured, failing which the performance aspects are to be severely compromised:

a) The non-coupler end needs to be thoroughly cleared and shall be free from any foreign material.

b) Use a clean rag or brush to lubricate the non coupler end with lubricant.

c) Clean and lubricate the coupler end of the pipe to be laid in similar manner.

d) Lubricate the exposed gasket in the same manner with pipe lubricant.

e) Keep the lubricated non-coupler end free from dirt, backfill material, and foreign matter so that the joint integrity is not compromised.

f) Push the coupler into non-coupler and align properly. Always push coupler end into non coupler end. For smaller diameter pipes simple manual insertion shall be sufficient. In every methodology, it should be ensured that the coupler end is adequately 'homed' within non-coupler end to ensure installation and tight joining seal. Therefore, prior to insertion always place a homing mark on appropriate corrugation of the non coupler end.

11.3.6.3 Jointing different pipe types or sizes

Sewerage/drainage system often encounters connecting pipes of different materials/sizes, etc. The fittings or adapters specifically designed for the purpose are available.
A selection of fittings designed to make the transition from one material directly to another are also available. In few cases, fitting may need to be used in combination with separate manufacturer's gasket or coupler to give proper effect to the transition.

11.3.7 Manholes and Catch Pit Connections

Brick masonry manholes can also be used at changes in pipe material, size, grade, direction and elevation. Manufacturer specified pre-fabricated appurtenant structures made of thermoplastic materials shall also be available for onsite user friendly installations. Similar methodology shall be followed for integration of catch pits.

11.3.8 Conduit Connections

Other connecting lines shall be integrated with the already laid system in the same manner as of original pipe lines.

11.3.9 Type of pipe material and jointing of storm conduits(rigid pipe)

11.3.9.1 Reinforced Cement Concrete Pipes (R.C.C Pipe)

The reinforced cement concrete pipes (IS:458-1988) are non-pressure pipes available under three classifications of NP_2, $NP_{3,}$ NP_4 That are commonly used in storm water conduits under appropriate loading conditions.

The R.C.C. pipes shall be laid in position over either concrete cradle or on the plain cement concrete bedding, 150 mm plain cement concrete (1:3:6) with carefully packed backfill of earth soil or dug material if suitable. The abutting faces of the pipes being coated by means of a brush with bitumen in liquid condition. The wedge shaped groove in the end of the pipe shall be filled with sufficient quantity of either special bituminous compound or sufficient quantity of cement mortar of 1: 3. The collar shall then be slipped over the end of the pipe and the next pipe butted well against the plastic ring by appliances so as to compress roughly the plastic ring or cement mortar into the grooves. Care being taken to see that concentricity of the pipes and the levels are not disturbed during the operation. Spigot and socket (S&S) R.C.C. Pipes shall be laid with pipe joints caulked with tarred gasket in one length for each joint and sufficiently long to entirely surround the spigot end of the pipe, The gasket shall be caulked lightly home but not so as to occupy more than a quarter of the socket depth, The socket shall then be filled with a mixture of one part of cement and one part of clean fine sand mixed with just sufficient quantity of water to have a consistency of semi-dry condition and a fillet shall be formed round the joint with a trowel forming an angle of 45 degrees with the barrel of the pipe. Rubber gaskets may also be used for jointing.

11.3.9.2 Cast –In Situ Reinforced Concrete Pipes

For conduit sizes beyond 2 m internal diameter cast-in-situ concrete sections shall generally be used, the choice depending upon the relative costs worked out for the specific project. The concrete shall be cast in suitable number of lifts usually two or three. The lifts are generally designated as the invert, the side wall and the arch.

11.3.10 Type of pipe material and jointing of storm conduits of Flexible pipe

11.3.10.1 HDPE Pipes

PE pipes are manufactured in three grades namely LDPE, MDPE, HDPE. HDPE pipes are commonly used in storm water conduits. HDPE pipes are manufactured in India conforming to (IS-4984-1995). They are available in standard length in6.0m and 12.0m.The installation of HDPE pipes should conform to IS 7634 part 2: 1975.Bedding materials may be dug materials, imported materials or as per design class of bedding given in foregoing section. The pipes are joined either in butt fusion welding, electro fusion welding or mechanical joints such as flange joints, telescopic rubber gasket joint, compression joint etc. Manufacturer’s jointing procedure may also be followed.

11.3.10.2 UPVC Pipes

These pipes are manufactured in India conforming to IS:4985-1988. They are available in standard length of 6.0m. Pipes are manufactured under various pressure classifications such as class 1, class 2, class 3, and class 4. Installation of pipe should conform to IS:7634 part 3: 1975. Bedding may of dug materials, processed granular materials or as per designed bedding as mentioned in foregoing sections.

Jointing of pipes may be carried out by three methods such as: 1) PVC solvent welded joints 2) Flanged joint 3) Push fit type rubber ring joint.

11.3.10.3 Glass fiber reinforced plastic pipes (GRP PIPES)

GRP pipes are now manufactured in India conforming to IS: 12709: 1994. Standard lengths of pipe in 6.0 m, 9.0 m and 12.0 m are available. Bedding may be dug materials, imported materials or as per design given in foregoing section. Jointing of GRP pipes is carried out by one of the following methods as per site requirement :

a) Socket and spigot gasket joint- provided with grooves either on the socket or in the spigot to retain an elastomeric gasket that shall be the sole element of the joint to provide water tightness.

b) Coupling joints- coupling with rubber gasket placed on each side are often used for jointing GRP pipes.

c) Mechanical coupling- Mechanical flexible couplings made of C/I, D/I, Steel are also used for GRP to GRP or GRP to other pipe joints.

11.3.10.4 Structured Wall PE pipe

The IS 16098 (Part I), IS 16098 (Part II) and EN 13476 also cover the performance requirements for the respective materials. These pipes are manufactured with externally corrugated wall configuration i.e. Double Wall (smooth inside layer & annular Corrugated outside wall) PE Pipes here in after called DWC PE Pipes. The pipes are integrated with coupler (socket) ends and joined through extremely user-friendly Push-fit jointing system without application of any foreign material.

11.3.11 Backfilling of the Trenches of Rigid pipes

Backfilling of the storm water conduit trench is a very important consideration in conduit construction. The method of backfilling to be used varies with the width of the trench, the character of the material excavated, the method of excavation and the degree of compaction required. In developed streets, a high degree of compaction is required to minimize the settlement while in less important streets, a more moderate specification for back fill may be justified. In open country it may be sufficient to mound the trench and after natural settlement return to re- grade the areas.

11.3.12 Construction of Backfill Envelope and Backfilling of the Trenches of Flexible pipe

These pipes and well compacted backfill envelope work together to support soil and traffic load.

In general, material used for construction of backfill envelop around the pipe comprises the following:

a) Initial backfill;
b) Side fill; and
c) Top backfill.

The material for backfill envelop shall be as per the structural design of flexible buried conduit. It can be the same material that were removed in the course of excavation or it can be fine sand/course sand/gravel depending on the over burden and superimposed load, but it should not be the concrete which invariably induces undesired rigidity in the system.

The remaining portion of backfilling shall be the materials that were removed in the course of excavation. These materials shall consist of clean earth and shall be free from large clod or stone above 75 mm, ashes, refuse and other injurious materials. After completion of laying of pipes, etc, first the backfill envelope shall be constructed as per design around pipe. Voids must be eliminated by knifing under and around pipe or by some other technique and compacted with necessary watering, either by hand rammers or compactors to a possible maximum level of proctor density.

Backfilling shall start only after ensuring the water tightness test of joints for the concerned conduit segments. However, partial filling may be done keeping the joints open. Precautions shall be taken against floatation as per the specified methodology and the minimum required cover.

11.3.13 Removal of sheeting or bracing

Sheeting driven below the spring line of a storm water conduit shall be withdrawn slowly at a time as the back-filling progresses. To avoid any damage to buildings, cables, gas mains, water mains, sewers etc. near the excavation or to avoid disturbance to the conduit already laid, portions of the sheeting may be left in the trenches.

11.3.14 Storm water conduit appurtenant Structures

Appurtenant structures besides of storm water conduit system consist Manholes, storm water inlet structures, siphons, flap gates, outfall structures etc. which are essential for the proper functioning of the storm drainage system. Therefore this section discusses these structures giving general description with specific emphasis on the features considered necessary for appropriate design.

11.3.14.1 Manhole

Manholes are openings constructed along conduit alignment whose primary function is to provide convenient access to the storm drainage system for inspection and maintenance. They also serve as flow junction and can provide ventilation and pressure relief to the storm drainage system. They are of several configurations like rectangular and circular type as illustrated in the figure 11.2 and 11.3. For large conduits access shafts are generally provided in circular shape and suitable size of openings to allow a workman with cleaning equipment without difficulty.

11.3.14.2 Construction of Manholes

The manholes shall be constructed simultaneously with the conduit line. The manholes shall have 20mm thick cement plaster in cement mortar 1:3 The foundation of manholes shall be 15cm thick cement concrete of appropriate grade and thickness may be increased to 30cm when subsoil water is encountered, the projection of concrete being 10cm on all sides of the external face of brick work. The floor of the manholes shall be in cement concrete of appropriate grade. Concrete half channel pipes of the required size and curve shall be laid and embedded in cement concrete base to the same line and fall as the conduit. Both sides of the channel pipes shall be benched up in concrete and rendered smooth in 20mm thick cement mortar and formed to a slope of 1 in 10 to the channel. Bricks on edge shall be cut to a proper form and laid around the upper half at all the pipes entering or leaving the manhole, to form an arch. All round the pipe there shall be a joint of cement mortar 12mm thick between the pipe and tile bricks. The ends of the pipes shall be built in and neatly finished off with cement mortar. The masonry shaft or the manhole shall be provided on the top with a heavy air tight cast iron frame and cover conforming to IS:1726 or any other approved type of frame and cover. Where the depth of the manhole exceeds 90cm below the surface of the ground, steps of cast iron or of any other approved material shall be built into the brick work. The distance between the two consecutive steps shall not be more than 40cm. The top at manhole shall be flush with the finished road level (IS:4111 Part I - 1967 Manholes).

The entire height of the manhole shall be tested for water-tightness by closing both the incoming and outgoing ends of the conduit and filling the manhole with water. A drop in water level not more than 50mm per 24 hours shall be permitted. In case of high subsoil water it should be ensured that there is no leakage of ground water into the manhole by observing the manhole for 24 hours after emptying it.

11.3.14.3 Location of Manholes

Manholes are constructed at every change in alignment, gradient and size as well as at the start of all conduits and branches and at every junction of two or more small size conduits. Nevertheless junction chamber is a special design of underground chamber used to join two or more large storm drain conduits. This type of structure is usually required where storm drains are larger than the size that cannot be accommodated by standard manholes. Junction chamber by definition do not need to extend to the ground surface, however it is recommended that riser structure be used to provide surface access.

11.3.14.4 Spacing of Manholes

Criteria for spacing of manholes have been developed in response to storm drain maintenance requirements. At a minimum, manhole should be constructed at the following points in storm drain system;

- Where two or more storm drains meet.
- Where pipe sizes change.
- Where change in alignment occurs.
- Where a change in grade occurs.

In addition manholes may be located at intermediate points along straight runs in accordance with the criteria mentioned in the following table. However individual agencies may have limitations on spacing of manholes due to maintenance constraints:

Table 11.5: Recommended Maximum spacing of Manhole

Pipe size (mm)	Recommended maximum spacing (mm)
300-600 diameter	40
700-1050 diameter	100
1000-1400 diameter	150
1500 and above diameter	300

Manholes should be constructed in accordance with standard drawings as required. Location of manholes in roadway reserves may be preferred as follows;

- Road side
- Median strips
- Centre of road pavement

11.3.14.5 Drop in Manhole

Where conduits of different characteristics are connected, transitions occur. The difference may be flow, area, shape, grade, alignment and conduit material with a combination of one or all characteristics. The vertical drop may be provided only when the difference between the elevations is more than 60 cm, below which it can be avoided by adjusting the slope in the channel and in the manhole connecting the two inverts. The following invert drops are recommended.

Table 11.6: Recommended invert Drop in Manhole

S.No.	Diameter	Invert drop
1	For conduits less than 400mm	half the difference in dia
2	400mm to 900 mm	2/3 the difference in dia
3	Above 900 mm	4/5 the difference in dia

11.3.14.6 Shape and Size of the Manhole

Manholes are constructed directly over the centre line of the conduit. They are rectangular, circular or square in shape. They should be of size that facilitate cleaning and inspection of conduits.

1. Rectangular Manhole

The minimum internal size of rectangular manhole between internal faces should be

- For depth less than 0.90 m, 900 mm × 800mm.
- For depth from 0.9 m and up to 2.5 m, 1200 mm × 900mm

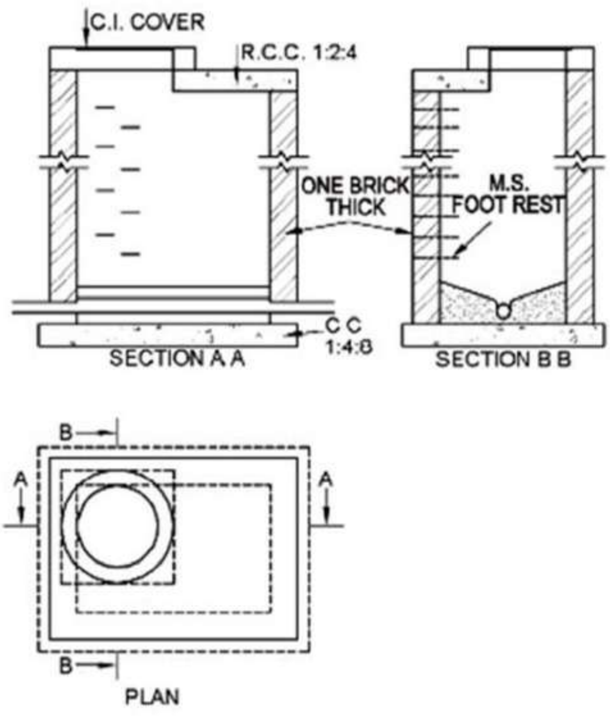


Figure 11.2: Rectangular Manhole

2. Circular Manhole

Circular manhole is stronger and preferred over rectangular or square manholes. These can be provided for all depths from 0.9 m upwards. They are constructed straight down in lower portion and slanted on top portion to narrow down the top opening so that it fits with the size of diameter of cover and frame as shown in the figure 11.3. The internal diameter of the circular manhole should be kept corresponding to the depth as follows:

- For depths above 0.9m and up to 1.65m – 900mm dia
- For depths above 1.65m and up to 2.30m- 1200mm dia
- For depths above 2.30 and up to 6.0m- 1500mm dia
- For depths above 6.0m and up to 9.0m – 1800mm dia

The manhole should be oriented in a manner so that workers enter into it while facing traffic.

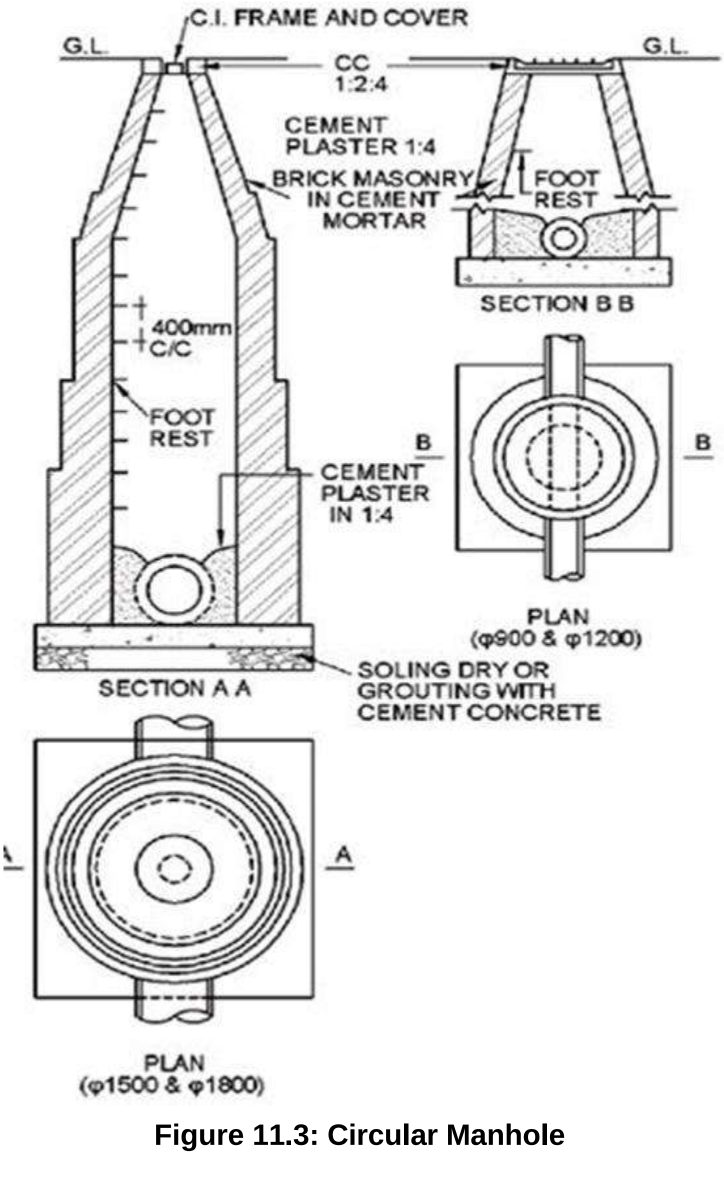


Figure 11.3: Circular Manhole

3. Drop Manholes

Drop manholes are used to connect storm water drains at significantly different levels and should be used where the level difference is greater than 600 mm.

The drop manhole can be provided by means of

a. Vertical drop in the form of a down pipe constructed inside / outside the well of manhole
b. Gradual drop in the form of cascade or ramp

A cascade is preferred for drain larger than 450 mm diameter. Downpipe are suitable for drains less than 400 mm diameter. When downpipes are used, the following recommendations are made:

a. Proper anchoring of the downpipe at the bottom in the form of 900 pipe bend surrounded by concrete should be provided
b. T branch at the top fitted with a flap valve inside the manhole should be made to avoid splashing.

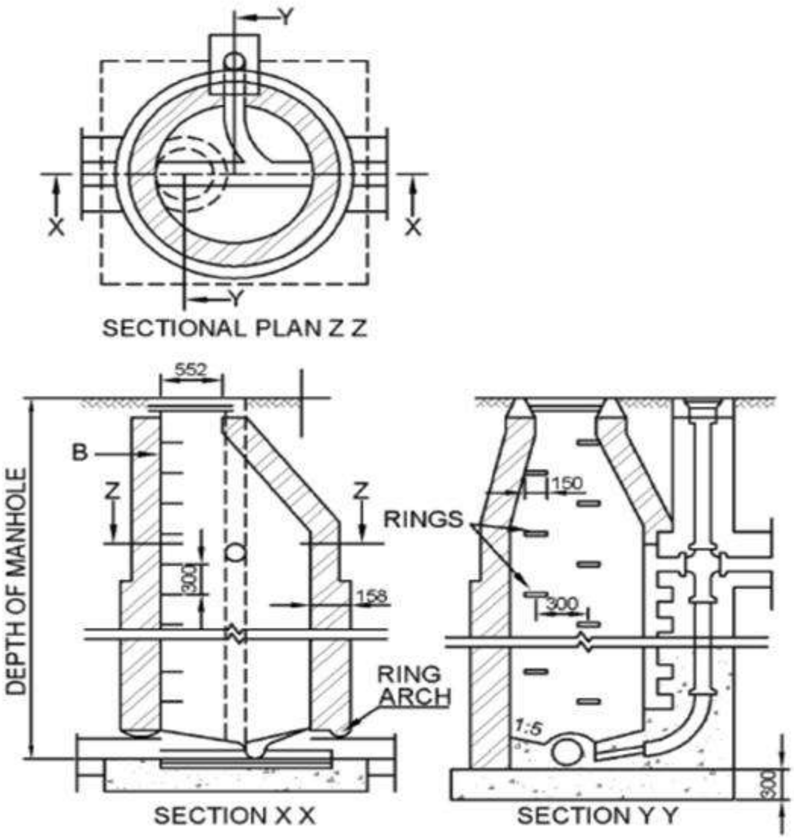


Figure 11.4: Drop Manhole

11.3.14.7 Cover and Frame of Manhole

Manhole cover and frame are designed to provide adequate strength to support superimposed loads, provide a good fit between cover and frame. For safeguarding against unauthorised tampering, the manhole cover should be bolted or secured with some locking mechanism. The size of the manhole should be such that there is a clear opening of not less than 560 mm diameter when cast iron cover and frame is used. They should confirm to IS 1726 (part 1-7).The frames of the manhole should be firmly embedded in correct alignment and level plain cement concrete on the top of the masonry. After completion of work manhole cover should be sealed by means of thick grease.

Heavy reinforced cement concrete cover with suitable lifting arrangement may also be used instead of C.I. manhole cover. Fibre reinforced plastic covers (FRP) may be used wherever such covers are available.

11.3.14.8 Access Steps in Manhole

Steps are provided for conveniently accessing the floor of the manhole for inspection and cleaning. The steps should be corrosion resistant. Steps coated with epoxy or fabricated from rust resistant material such as stainless steel or aluminium coated with bituminous paint are preferable. Steps made from reinforcing steel are not recommended. It is suggested by certain agencies to eliminate the use of steps with reason to avoid the danger of rust damages steps as well as unauthorised access to manhole. Besides, it is said that maintenance personnel shall use their own ladder for inspection or cleaning. Spacing of the steps should be maintained approximately 300-400mm and should be fixed staggered in order to have ease in ascent and descent. Cat ladder should be used in manhole deeper than 4-25m or where manhole is frequently entered. Step iron and ladder should start at not more than 600mm below cover level and continue to benching.

11.3.14.9 Intermediate Platform

Manhole deeper than 4.25 m from the cover level should be provided with intermediate platforms at regular intervals. The headroom between platforms should not be less than 2.0 m. The size of the platforms should be 800 mm × 1350 mm. The platform should be fitted with handrail and safety chains at the edge to protect workers against falling down.

11.3.14.10 Inverts and benching

The inverts should be curved to the radius of the pipe and carried up in flat vertical surfaces and should match the cross sections & bends and gradient of the respective storm water drains. The benching should be plain surface sloping gently downward towards the drains. A gradient of the benching of 1 in12 may be provided. The socket end of the pipe should be cut off and should not project inside the manhole.

The crown of the incoming and outgoing conduit should be kept at same level and necessary slope should be given in the invert drain of the manhole chamber preferably 1 in 10.The manhole should be safeguarded against uplift ground water pressure as well as against entry of ground water. U shaped small channels should be constructed integrally with concrete base of the manhole chamber to carry the flow in conduit. The side of channel should be kept equal to the diameter of the largest conduit. Where more than one conduit enters the manhole the channel should be smoothly curved to carry adequately the peak flow. The pipe joints should be kept outside the manhole chamber and the inlet and out let pipe should be made flush with internal face of the manhole chamber. The inlet and outlet pipes built with the wall of the manhole should be properly protected with cement concrete cover round the pipes against crushing of wall loads. Inside and outside brick surface of manhole should be plastered 1:3 cement mortar and inside surface should be finished smooth with neat cement punning. Whenever sub-soil water is encountered a rich mix of plaster with water proofing chemical compound may be used.

11.3.14.11 Outfall Structure

Outfall conduit is supported with a brick wall generally of 425 mm thick in 1:3 cement mortar at the point of disposal to a river or stream. The outfall conduit should project 100 - 150 mm inside the bank of river / stream. To protect the bank against erosion, it is necessary to pave the bank 2 m on either side from the point of disposal with cement concrete block providing toe wall and apron to safeguard against slipping of revetment as well as the erosion of the bed of the river / stream.

7.1 Inverted Syphon

Inverted siphon or depressed pipe which should stand full even without any flow and shall run with pressure above atmosphere on account of being depressed below hydraulic grade line . Its purpose is to carry the storm water flow under an obstruction such as a stream or depressed highway and to regain the permissible elevation after crossing the obstruction to maintain gravity flow or pumping whichever is feasible. Siphons can consist of single or multiple barrels however it is recommended that a minimum of two barrels should be provided as shown in the fig.11.5.

Following criteria may be considered in designing siphons

a) Self-flushing velocities should be provided under a wide range of flows.
b) Hydraulic losses should be minimized.
c) Provisions for cleaning should be made.
d) Sharp bends should be avoided.
e) the rising portion of the siphon should not be made too steep as to make it difficult to flush deposits
f) There should be no change in pipe diameter along the length of the siphon.
g) Provision for drainage should be considered.
h) Head should be sufficient to cover the entry, exit and friction losses and should develop not less 1.0 m/s self-cleansing velocity.
i) Inlet and outlet chamber should have sufficient room for entry for cleaning and maintenance of siphons.
j) Provision should be made for isolating the individual pipe of the siphon to facilitate cleansing.
k) Proper by pass arrangement should be provided for inlet chamber.

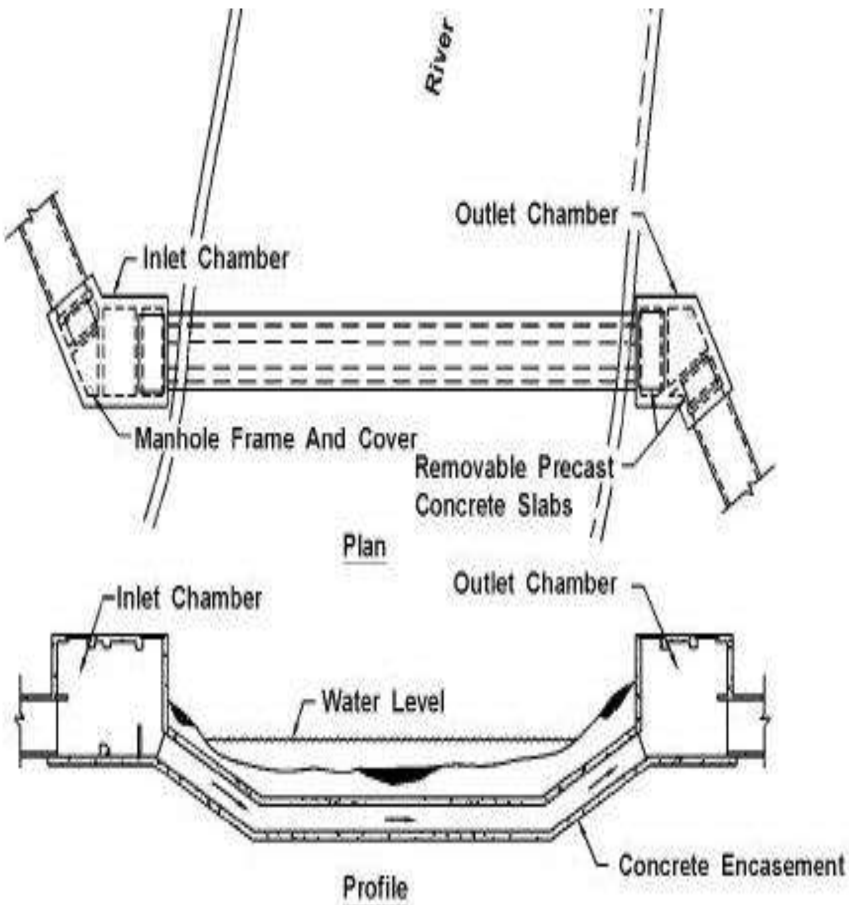


Figure 11.5: Inverted siphon with minimum two barrels

11.3.14.12 Over Flow Device

At times it is needed to separate sewage flows from storm water either from a channel or pipe in order to divert the sewage flows to treatment plants instead of disposing with storm water that may cause hazardous pollution problem. Combined sewage systems are generally equipped with such overflow device to get rid of heavy storm water flow during wet seasons.

1. Leaping Weir

Leaping weir is the most common device that is formed by gap in the invert of a sewer through which the dry weather flow or sewage falls and over which portion of all storm water leaps over to overflow pipe. Leaping weirs have the advantage of operating as regulator without moving parts but they offer the disadvantage of depositing grit in the low flow channel. However it is desirable to design the weirs with moving crests to make the opening adjustable.

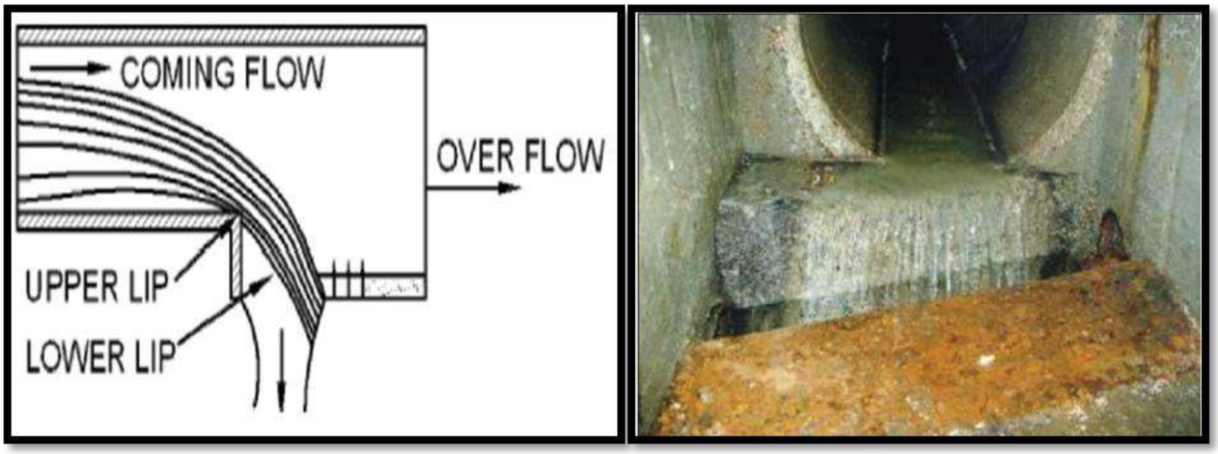


Figure 11.6: Leaping weir

2. Intercepting Conduits

The above device is invariably used to intercept sewage from storm flows and convey through intercepting conduits to waste water plants for treatment.

Delhi Jal Board has recently executed intercepting sewer project to intercept sewage from storm water drains flowing to Yamuna River in order to control heavy pollution of the river.

3. Flap Gates and Flood Gates

Flap gates are installed at or near storm drain outlets for the purpose of preventing back flooding of the drainage system at the high tides or high surges in the Receiving Ocean or tidal streams. A small differential pressure on the back of the gate is kept so that the flap should open at a very small head differential. Flap gates are typically made of cast iron, or rubber or steel and available in round square and rectangular opening and in various design and sizes. Adequate storage may be necessary if pumping is to be avoided during closure of the gate to prevent back flow in case of the high tide. To control this flow from the storage tank, Flap gates or penstocks are provided which can be opened and closed quickly at the predetermined stages of high rise of river or sea level. The gates generally have electrical drive mechanisms. Flap gate are usually hinged by key and lock type arrangement that makes it possible to get the gate shutters seated firmly. Hinge pins, links etc. should be of corrosion resistant materials.

11.4 Storm water drain

Storm water drains are surface drains which are constructed as open or covered drains with suitable gradient to carry the storm water flows from the catchment to the safe disposla point. Drainage in the urban context is classified as given below:

Tertiary drains: In urban catchments, tertiary drains collect storm water from sub-zones and convey to the secondary drains.

Secondary drains: These drains collect storm water from tertiary drains and zones. They discharge the storm water into the primary drains.

Primary Drains: In urban catchments, primary drains are main drains that collet storm water from secondary drains and discharge to the safe disposal point.

11.4.1 Construction of Storm Water Drains

This section discusses the construction of surface drains such as tertiary, secondary and primary drains. The tertiary drains are generally small drains that are constructed in rectangular section whereas; secondary and primary drains are larger drains that are normally constructed in trapezoidal section.

General preparation of works as given in section 11.3 should be followed as applicable. Drains are generally either of masonry or RCC construction. The general construction description of masonry and RCC drains are given as follows:

1. RCC drains

Tertiary drains are usually constructed in rectangular section either of masonry or reinforced cement concrete.Where it is proposed to construct pre cast RCC drain, the same should not be less than 50mm thick and should be reinforced with 3 longtudinal bars of 6mm diameter and 2 cross bars of same size in 0.6 m length and mould should be removed after 48 hours then they shall be kept well watered for a fortnight and after this watering shall be discountinued and the drain should be left to cure for another fortnight before laying. The ground should be kept to the exact shape and slope at which drains are to be laid and the trench will be watered and rammed.

2. Brick Drains

Brick drains are constructed in brick work, also. The brick work shall be in cement mortar 1:3 and plastered smooth with cement plaster of 1:2,20 mm thick. A change in the alignment of the brick drain shall be on suitable curve confirming to the surface alignment of the road.

11.4.2 Rectangular Section

In congested urban areas, small or medium drains are constructed in rectangular section covered with suitable RCC slabs to protect against dumping of solid waste from the local residents. Rectangular drains are normally constructed in hilly regions due to space crunch.

11.4.3 Trapezoidal section

Primary and secondary drains that normally carry considerable quantity of storm flows are constructed in trapezoidal section. Especially outfall channels that sometimes carry entire storm flows from the catchment are designed in larger sections that often resemble irrigation channels. In such cases it is preferable to economize the cost by constructingearthen channels with cement concrete lining.

11.4.4 Kerb and Gutter

Gutters are provided at both edges of pavement all along the length for collecting rain water from the pavement. They are constructed in triangular section and are generally in RCC. The details of gutter design are given in Chapter 5.

11.4.5 Construction Procedures of Storm Water Drains

The construction of storm drains is described as follows.

Step1: Marking of Alignment - The surveyor marked out the alignment for the trench to be dug.

Step 2: Digging/Excavation - The surveyor marks out the depth and width of the trench as per size and design requirement to be excavated with a mechanical excavating machine.

Step 3: After excavation concrete blinding is done - Blinding is done on the surface area in order to correct any irregularities in level of the bed of the excavated surface, and to provide a smooth, level and regular surface to receive the concrete base. It's usually ±50 mm depending on the rate of regularity of the surface area. The blinding is a mass concreting and it's advisable to spread to cover entire width of the excavated trench.

Step 4: Reinforcement (iron Rod) positioning - Reinforcement (spaced as designed) is placed into position on the blinded surface but positioned at the centre of the base with the aid of concrete biscuit to create a concrete cover.
There should be concrete cover between the reinforcement and the base of the drain concrete base and the top of the drain wall.

Step 5: Laying of concrete base on the blinded surface and the positioned reinforcement - A guiding panel is placed into position to guild in the laying of the concrete base in order to achieve a uniformly alignment base edge, thickness and width, also to manage concrete material while pouring. The base is cast with the U shape reinforcement bottom in between the concrete base achieving concrete cover below and above.

Step 6: After setting and drying of the concrete base, next is to position the side wall panel form work - The floor base is marked to give the required internal width where the panel will be positioned. The panel wall spacing and wall height is as per requirement of design; the panel is lubricated, clipped and prepared to accept the Concrete. After casting, and setting, the panel is removed and concrete cured.

Step 7: In order to avoid settlement, back filling and compaction of the back filling should done immediately

11.4.6 Covering of Drains

Secondary and tertiary drains constructed in congested sectors of the urban area should be covered with precise RCC slabs of suitable size wherever needed. RCC Slabs in smaller lengths capable of lifting by 1 – 2 persons are pre cast with lifting hooks. After proper curing these slabs are placed over the drain and joined with cement plaster. When the drains are required to be cleaned, these slabs can be removed easily at suitable intervals and cleaning operation can be done. Even secondary and primary drains of larger section, it will be uneconomical to cover them instead they can be fenced along their edges or small parapet may be constructed to protect children or men falling in them. However, if resources permit ULB may undertake to cover such drains if it is deemed expedient in favour of public welfare.

11.4.7 Box Drains

RCC box drains are constructed along drainage reserve of heavy vehicular traffic. These drains are designed to withstand vehicular load and carry the large storm water volume to the safe disposal point. Street inlets are provided between 15 – 30 m interval in order to admit storm water in the box drain.

Figure 11.7: Box drain

They are laid 200 – 300 mm below ground level in suitable gradient having access holes at interval of 30 m to facilitate cleaning etc. A typical figure of a box drain is shown in the figure 11.7.

11.4.8 Safety and Social Safeguard

The need for safety precautions in any specific project area must be recognized and observed before and during construction activities. Following care should be taken:

1. Any construction will draw on-lookers, especially children. Onlookers should be kept away from the operating equipment and from the edges of excavations.
2. Traffic must be diverted and or controlled at all times unless permission has been received from the proper authority to completely close a road or divert the traffic.
3. Emergency vehicles must not be delayed.
4. Vehicular access to homes and places of business should be maintained. If this is not possible, the occupant should be apprised of the situation by the Contractor or the Engineer. It is an absolute necessity that good relations be maintained with the general public.
5. When leaving the project at night, no unnecessary obstructions to traffic should be left behind, such as earth lumps from the trench excavation or sections of pipe that encroach on the roadway.
6. All necessary barricades for the construction close to traffic need to be made.
7. Provision of warning signs 150 meters in advance of any place on the project where the operations interfere with the use of the road by crosses or coincides with an existing road.
8. The construction area of the project should be properly lighted.

a) Signs, Signals and Barricades

- Before starting any job in a street or other traffic area, study the work area and plan your work.
- Traffic may be warned by high-level signs well ahead of the job site.

- Traffic cones, signs or barricades to be arranged around the work, and signboards to direct the traffic.
- Whenever possible place your work vehicle between the working site and the oncoming traffic.
- Use fluorescent jacket while working along roads.
- Construction area should be barricaded so that unaythorized persons especially children may not enter within the construction site. Light signals should be placed also during night time.

11.4.9 Completion of Works

1. The administrative Department/Ministry shall be kept informed at regular intervals about the stages of progress of work so that the client's observations, if any, could be responded to before the work is completed.
2. On completion of the work, the Administrative Department/Ministry should be intimated of the same and formal handing over arranged in writing. Reasonable advance intimation of completion of the work should be given to the concerned Department to enable them to make arrangements for taking over.
3. Completion plans of the project, including all services, should be prepared and submitted along with the completion report showing the expenditure incurred on the project.

11.4.10 Procedures for Handing Over

On satisfactory completion of works a joint inspection should be carried out to ensure that works are completed in accordance with the standard design and maintenance requirement laid down in this manual. Before issue of completion certificate, a handing over inspection report should be submitted ensuring that all outstanding works are completed. Within three months of issue of completion certificate and prior to the end of maintenance period a joint inspection should again be carried out to check if further works are required and that all outstanding or remedial works have been completed. Besides, during the planning and design stages a design memoranda should be prepared so that design parameters, handing over requirement or partial handing over arrangements of large projects can be agreed by maintenance authorities. If unforeseen problems are encountered during construction and changes have to be made, the maintenance authority should be consulted so that the changes may be incorporated. On completion changes made should be incorporated in the design memorandum before handing over charge of completed works. Reference may also be made to project administrative procedures and the relevant technical memoranda if any, for details of handing over and taking over procedures.

11.4.11 Procedure for handing Over in Dry Conditions

All conduit lines, channels and culverts, etc. to be handed over should be inspected in dry conditions wherever possible. In the case where the pipes, culverts or channels have to be commissioned prior to handing over (e.g. due to the requirement to maintain the existing flow or staged completion) and a temporary diversion of flow is not feasible, an additional inspection should be arranged prior to the commissioning. In certain circumstances, closed circuit television (CCTV) survey of the pipes and internal faces of the manholes showing each connection pipe before commissioning can be adopted as an alternative to the joint inspection but prior agreement with the respective operation and maintenance authorities may be obtained.

11.4.12 Handing over Drainage Records

After handing over the works as per procedures outlined, the following documents should be submitted as soon as possible, but not later than 3 months under any circumstance:

- As-built drawings, in hard-copy and electronic format, if applicable.
- Hydraulic and structural design calculations, in electronic format, if available.
- Construction records including major acceptance tests, material quality records, product specifications and warranties.
- O & M manual and system manual.
- Maintenance manual for slope embankment.

In the event that as-built drawings are not available at the time of the handing over inspection, marked up prints of the working drawings showing the final amendments and the extent of works to be handed over should be provided. Records of material quality and acceptance tests should also be available for scrutiny.

APPENDICES

APPENDIX A 2.1: CHECKLIST FOR SUBMISSION & SCRUTINY OF DETAILED PROJECT REPORT

(STORM WATER DRAINAGE) (SWD)

(to be filled in and certified by the highest city –level Officials, both technical and administrative, such as Chief Engineer/City Engineer/ Municipal Commissioner)

Instructions:

1. The DPR shall be formulated as per the guidelines given in Manual of Storm Water Drainage Systems published by the Ministry and as per the Department procedures.
2. DPR shall be technically sanctioned by the Competent Authority the State Govt./ULB before forwarding it to the Ministry.
3. Each and every page has to be signed at the bottom by the officials.
4. Each field has to be filled in appropriately as 'yes', 'no', 'not required', 'not done', 'not used' etc. No field has to be left blank. Give explanatory comments wherever 'no' is indicated.
5. Non- definite entries such as 'will be done later', 'will be furnished later' etc. will not be accepted.

CERTIFICATE:

This is to certify that that the undersigned have read the contents of the check list fully and have responsibly made the entries true to the best of knowledge and understanding. In case the information furnished in the check list enclosed is found to be incorrect for any reason, whatsoever, the undersigned may be held liable for disciplinary action as per applicable Government rules.

Certified that

(i) The designs and drawings have been approved by the Competent Authority.

(ii) The detailed estimates and cost estimates are as per the current schedule of rate and/or rate analysis and latest pro-forma invoices (current market rates).

(iii) The DPR has been technically sanctioned by the Competent Authority in the State Govt./ULB.

Signed: Signed:
Name: Name:

CHECKLIST FOR SUBMISSION & SCRUTINY OF DPR

(STORM WATER DRAINAGE SYSTEM)

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
1.	**GENERAL COMPONENTS**	
1.1	Name of the town/city/District/State for which scheme has been formulated with name of the scheme (a) Name of the City/Town: (b) Name of the District: (c) Name of the State : (d) Name of the Scheme:	
1.2	Date of DPR appraised by State Level Nodal Agency (SLNA) and whether a copy of appraisal report (duly authenticated by the competent authority) has been forwarded with DPR. (a) Date of appraisal: (b) Name of the appraisal agency: (c) Original Estimated cost: (d) Appraised cost: (e) Major comments/observations made by appraisal agency.	
1.3	Whether the commitment to launch the scheme immediately after approval of Govt. of India / Administrative approval of the scheme is appended in DPR.	
1.4	(a) Whether Project formulation justification (need for the project) has been furnished in DPR. Please justify the need of the project. Justification: (b) Whether executive summary of the project is furnished in the DPR	

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
1.5	Whether linkages of this scheme have been established with other ongoing STORM water drainage schemes being funded by the Central/State Govt./other agencies, if any. Please furnish the details.	
1.6	Whether the map showing administrative and political jurisdiction of the project area has been given in DPR. Area within Municipal limit : ……… sq.km. Extent of area considered in the DPR : ……….sq.km. Additional area (beyond Municipal limit) considered in the DPR and justify the reasons: ……..sq.km	
1.7	Whether the land use pattern of the city / town / project area as per the approved Master Plan has been given in DPR.	
1.8	Whether the DPR including the design, drawings, cost estimates, analysis of rates has been authenticated by Competent Authority of State Govt./ ULB and Quasi-Technical sanction of DPR / Technical & Financial Verification Certificate has been attached with DPR	
1.9	In case any proposed pumping main for storm drainage lines is crossing Railway line/ Highway & their bridge (wherever applicable), whether the clearance from concerned authority such as State Pollution Control Board (SPCB), Highways, PWD, Railways has been obtained and copies of the permission and their estimate for the same has been provided in DPR. If not, the present status of action initiated may be furnished below.	
1.10	Whether the provision for separate electric feeder line to the storm water pumping stations (to take care of frequent power failure and voltage fluctuation problem) from HT line and an agreement between Electricity Department and Urban Local Bodies (ULBs) has been furnished in the DPR	
1.11	Whether the commitment from Electricity Department for un-interrupted power supply (for pumping stations) is obtained	
1.12	Whether the topographic map of the city/town/project area to the scale has been given in DPR/Zone wise maps to scale showing all streets.	
1.13	Whether soil investigation report – bore hole logs at least at a grid of 1 km x 1 km or Geological Survey Data has been forwarded with DPR.	

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
1.14	Whether Contour map of the project area has been annexed with the DPR.	
1.15	Whether resolution from the ULB for meeting the regular expenditure on O&M of the storm water drainage system is enclosed in DPR.	
2. ENGINEERING COMPONENTS		
2.1	Storm water drainage network detailing	
	Total length of drain & other infrastructure (Total length and drains which are in good condition and can be integrated with proposed planned drainage system): Tertiary drain : ....Km (total) ....KM (drains in good condition) Secondary drain : ....Km (total) ....KM (drain in good condition) Primary drain : ....Km (total) ....KM (drain in good condition) SWD Pumping Stations: Nos....... Capacity of Pumps............Length of Pumping Mains........... Km	
	Proposals for Rehabilitation Tertiary drain : ....Km Secondary drain : ....Km Primary drain : ....Km SWD Pumping Stations: Nos....... Capacity of Pumps............Length of Pumping Mains........... Km	
	Proposals for new construction Tertiary drain : ....Km Secondary drain : ....Km Primary drain : ....Km SWD Pumping Stations: Nos....... Capacity of Pumps............Length of Pumping Mains........... Km	
2.2	Total length of road : ....Km	
2.3	Please furnish various project components (major components)	

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
2.4	Project Area and population (i) Please furnish the details of city/project area, (a) Area of the town/city (municipal limit): ………………Sq. km (b) Extent of the project area considered in the DPR: ………………..sq. km (c) Additional Area(beyond municipal limit) considered in the DPR:……sq.km (d) No. of Households (as per 2001 and 2011 census): (ii) Whether population projection has been adopted as per CPHEEO Manual and given in DPR (a) City population As per 2001 Census :…………..lakhs As per 2011 Census : ……………lakhs Initial stage : ……….. lakhs +floating population (if any)----------lakh (…………….AD) Intermediate stage : ……….. lakhs+ floating population (if any)----------lakh (………………AD) Ultimate stage : ……….. lakhs+ floating population (if any)----------lakh (………………AD) Population growth rate adopted: ………. %/ year (based on the past 5-6 decadal growth rate) Demographic Method adopted and justification : (b) Whether the population projection has been made in consonance with the Developmental Master Plan (c) Project Area	

<table>
<tr><th>S. No</th><th>Description</th><th>Write ‘Yes’ or ‘No’ etc. in the column below

If Yes, give Page No./DPR volume reference. If No, reasons thereof</th></tr>
<tr><td></td><td>Initial stage : ………… lakhs
Intermediate stage : ………… lakhs
Ultimate stage : ………… lakhs
Population growth rate adopted: ………. %/ year
(based on the past 5-6 decadal growth rate)
(d) No. of wards (within municipal limit) : …………</td><td></td></tr>
<tr><td>2.5</td><td>Whether the development master plan with land use pattern, identification of existing and future roads/streets, water bodies such as lakes and ponds, natural drains and rivers has been furnished for the urban agglomeration</td><td></td></tr>
<tr><td>2.6</td><td>If yes, give the master plan year.
If no, give present status of master plan preparation;</td><td></td></tr>
<tr><td>2.7</td><td>Land use patterns, present and proposed preferably on shape file format
<table>
<tr><td colspan="2" rowspan="2">Land Use</td><td colspan="2">Master Plan</td><td colspan="2">City/ULB Area</td><td colspan="2">Project Area</td></tr>
<tr><td>Present Master Plan: Year ....</td><td>Proposed Master Plan: Year ....</td><td>Present Area .... (Year ....</td><td>Proposed Area .... (Year ....</td><td>Present Area .... (Year ....</td><td>Proposed Area .... (Year ....</td></tr>
<tr><td rowspan="2">Total Area</td><td>Hectares (Ha)</td><td>.........</td><td>.........</td><td>.........</td><td>.........</td><td>.......</td><td>.........</td></tr>
<tr><td>%</td><td>100%</td><td>100%</td><td>100%</td><td>100%</td><td>100%</td><td>100%</td></tr>
<tr><td rowspan="2">Residential area</td><td>Ha</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>%</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Area under Roads>3m wide</td><td>Ha</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>%</td><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table></td><td></td></tr>
</table>

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof

Area under Roads & streets <3 m wide	Ha						
	%						
Markets (wholesale, vegetable, grain, other	Ha						
	%						
Area under Railways, Airports	Ha						
	%						
Institutional Area	Ha						
	%						
Industrial Area	Ha						
	%						
Green, open, park, agricultural area	Ha						
	%						
Lakes, Ponds	Ha						
	%						
Natural drains, sub-drain, nallahs, rivers	Ha						
	%						

Give Coefficients of Imperviousness adopted for design for various land uses:

Land use classification	Coefficient of Imperviousness as per Manual / Derived	Coefficient of Imperviousness as per DPR
Residential	0.60 to 0.75	
Roads, paved surface of footpaths	1.00	
Commercial	0.70 to 0.90	
Paved markets	1.00	
Unpaved markets	0.40 to 0.70	
Mixed type markets	0.40 to 0.90	
Mixed Development	0.60 to 0.90	
Industrial	0.60 to 0.90	

<table>
<tr><th rowspan="2">S. No</th><th rowspan="2">Description</th><td>Write ‘Yes’ or ‘No’ etc. in the column below</td></tr>
<tr><td>If Yes, give Page No./DPR volume reference. If No, reasons thereof</td></tr>
<tr><td></td><td>
<table>
<tr><td>Institutional</td><td>0.60 to 0.90</td><td></td></tr>
<tr><td>Large establishments</td><td></td><td></td></tr>
<tr><td>PSUs</td><td>0.60 to 0.90</td><td></td></tr>
<tr><td>Railways</td><td>0.60 to 0.90</td><td></td></tr>
<tr><td>Airports</td><td>0.60 to 0.90</td><td></td></tr>
<tr><td>Lakes, ponds</td><td>1.00(considering FSL)</td><td></td></tr>
</table>
</td><td></td></tr>
<tr><td>2.8</td><td>List out all natural drains in the city/project / master plan area. Give the names (IDs)and length
Natural storm water drains preferably on GIS maps (use additional sheets if required):
<table>
<tr><th>S No</th><th>Name / ID</th><th>Length, Km</th></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td></tr>
</table>
</td><td></td></tr>
<tr><td>2.9</td><td>Give width-wise detailing of natural storm water drains(use additional sheets if required):
<table>
<tr><th>S No</th><th>Width</th><th>Length, Km</th></tr>
<tr><td></td><td>Upto 2m</td><td></td></tr>
<tr><td></td><td>>2m upto 5m</td><td></td></tr>
<tr><td></td><td>>5m upto 10m</td><td></td></tr>
<tr><td></td><td>>10m upto 30m</td><td></td></tr>
<tr><td></td><td>>30m(give further widths if necessary)</td><td></td></tr>
</table>
</td><td></td></tr>
</table>

<table>
<tr><th>S. No</th><th>Description</th><th>Write ‘Yes’ or ‘No’ etc. in the column below
If Yes, give Page No./DPR volume reference. If **No**, reasons thereof</th></tr>
<tr><td>2.10</td><td>Whether the storm water drainage network has been divided into basins, sub-basins, catchments and overlaid on the development master plan? Give details.</td><td></td></tr>
<tr><td>2.11</td><td>Demarcating of zones and subzones as per the map of the project area (use additional sheets if required):
<table>
<tr><td>Whether the Master Plan Area/Project Area has been divided into catchments and sub-catchments for Storm Water Management</td><td colspan="3">Yes/No</td></tr>
<tr><td>Total no. of catchments (storm water drainage Zones)</td><td colspan="3"></td></tr>
<tr><td>Name/No. of catchment (zones)</td><td>1</td><td>2</td><td>3 etc</td></tr>
<tr><td>Area under catchment (various zones), Ha.</td><td></td><td></td><td></td></tr>
<tr><td>No. of sub-catchments (sub-zones) under each zone</td><td></td><td></td><td></td></tr>
<tr><td>Describe boundaries of each catchment (use separate pages) Ridge/Road/Rly. Line etc.</td><td colspan="3"></td></tr>
<tr><td>Give name/no. of each sub-catchment, its boundaries and arial extent (use separate pages)</td><td colspan="3"></td></tr>
<tr><td>Give land-use classification for each catchment and sub-catchment with totals ((use additional sheets if required))</td><td colspan="3"></td></tr>
<tr><td>Whether Catchment areas which are out of municipal limit likely to contribute in the project area has been taken into account</td><td colspan="3"></td></tr>
</table></td><td></td></tr>
<tr><td>2.12</td><td>Details of each sub-catchment (use additional sheets if required):
<table>
<tr><td>Name/ID No of sub-catchment</td><td></td></tr>
<tr><td>Total area</td><td></td></tr>
<tr><td>Define boundaries</td><td></td></tr>
<tr><td>Land use classification</td><td></td></tr>
<tr><td>Area under...... Residential</td><td></td></tr>
<tr><td>..........Roads etc.</td><td></td></tr>
</table></td><td></td></tr>
</table>

S. No	Description		Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
	...........Institutional		
	...........Industrial		
	...........Lakes/Ponds		
	...........Any other (add rows)		
	Total of above		
	Name/ID of main drain of sub-catchment		
	Total length of main drain		
	Width-wise length of main drain (proposed)		
	<2m		
	>2m – upto 5m		
	>5m- 10 m		
	>10m-30m		
	>30m		
	Total of above		
	Whether boundary of main drain demarcated and protected	Yes/No	
	Length of main drain protected		
	Length of main drain not protected		
	Action, if any for full protection		
	Whether drain outfall free or obstructed?		
	Invert level of drain outfall		
	Invert at outfall		
	at + 30m		
	at +60m		
	at +90m		
	at +120m : etc		
	Storm water disposal body HFL		

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
	<table><tr><td>Normal water level Bed level</td><td></td></tr><tr><td>Whether drain trained/untrained</td><td></td></tr><tr><td>Trained length</td><td></td></tr><tr><td>Untrained length</td><td></td></tr><tr><td>Any constrictions like culvert</td><td></td></tr><tr><td>Identify each such culvert</td><td></td></tr><tr><td>Drain -- Bed surface material & condition</td><td></td></tr><tr><td>Manning's 'n' value</td><td></td></tr><tr><td></td><td></td></tr><tr><td>Sidewalls material & condition</td><td></td></tr><tr><td>'n' value</td><td></td></tr><tr><td>Combined 'n' value at every multiple o.1 m depth of flow</td><td></td></tr></table>	
2.13	Coefficient of Roughness for use in Manning's Formula: (in the DPR column, fill values only for the material used and mark others as 'not used') <table><tr><td></td><td colspan="2">Type of Material</td><td>'n' as per Manual</td><td>'n' as per DPR Design</td></tr><tr><td rowspan="2">1</td><td rowspan="2">Salt glazed Stoneware Pipes</td><td>a) Good</td><td>0.012</td><td></td></tr><tr><td>b) Fair</td><td>0.015</td><td></td></tr><tr><td rowspan="2">2</td><td rowspan="2">Cement Concrete Pipes(with collar joints)</td><td>a) Good</td><td>0.013</td><td></td></tr><tr><td>b) Fair</td><td>0.015</td><td></td></tr><tr><td>3</td><td colspan="2">Spun Concrete Pipes (RCC & PSC) with socket & spigot joints (Design value)</td><td>0.011</td><td></td></tr><tr><td rowspan="3">4</td><td rowspan="3">Masonry</td><td>a) Neat Cement Plaster</td><td>0.018</td><td></td></tr><tr><td>b) Sand & cement plaster</td><td>0.015</td><td></td></tr><tr><td>c) Concrete –steel troweled</td><td>0.014</td><td></td></tr></table>	

<table>
<tr><th>S. No</th><th>Description</th><th>Write ‘Yes’ or ‘No’ etc. in the column below
If Yes, give Page No./DPR volume reference. If **No**, reasons thereof</th></tr>
<tr><td></td><td>

		d) Concrete – Wood troweled	0.015	
		e) Brick in good condition	0.015	
		f) Brick in rough condition	0.017	
		g) Masonry in bad condition	0.020	
5	Stone Work	a) Smooth dressed Ashlar	0.015	
		b) Rubble set in cement	0.017	
		c) Fine, well packed gravel	0.020	
6	Earth	a) Regular surface in good condition	0.020	
		b) In ordinary condition	0.025	
		c) With stones and weeds	0.030	
		d) In poor condition	0.035	
		e) Partially obstructed with debris or weeds	0.050	
7	Steel	a) Welded	0.013	
		b) Riveted	0.017	
		c) Slightly tuberculated	0.020	
		d) With spun cement mortar lining	0.011	
8	Cast Iron	a)Unlined	0.013	
		b)With spun cement mortar lining	0.013	
9	Asbestos Cement		0.011	
10	Plastic (smooth)		0.011	

</td><td></td></tr>
<tr><td>2.14</td><td>Whether the authenticated data of autographic rainfall data for the project area for the last 25 to 30 years or more has been obtained from India Meteorological Department and furnished in the DPR? Whether it has been analysed as described in the CPHEEO Storm Water Drainage Manual and the intensity – duration – frequency (IDF) curve for the project area has been drawn? Give details as per the model below:</td><td></td></tr>
</table>

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
2.15	Rainfall Data & Analysis (use additional sheets if required):	

No. of years of autographic rainfall Data from IMD(India Meteorological Department)	
Whether autographic rainfall data analysed and arranged in duration (minutes) and intensify (mm/hr)	
Duration-wise compilation of rainfall data (refer Manual)	
Frequency of storms of different duration	
Total no. of rainfall events of 5 min duration (arranged in ascending intensity)	
Similarly, events of 10 min duration (arranged in ascending intensity)	
Similarly, events of 15 min duration	
20 min duration	
30 min duration	
40 min duration	
60 min duration	
90 min duration	
120 min duration	
150 min duration	
180 min duration, etc	

Storm Frequency (or Storm Return Period / Flooding design interval):

Land Use Classification	Storm frequency as per Manual	As per DPR Design
a)Residential Areas		
i) Peripheral areas	Twice a year	
ii) Central and comparatively high priced areas	Once a year	
b)Commercial and High-priced areas	Once in 2 years	

S. No	**Description**	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
	Analysis of Frequency of Storms (Rainfall Events)(Historical data)	

Duration of rainfall, in minutes	No. of storms of particular duration of the intensity(mm /hr) given below or more during the data period							
	20	30	35	40	45	50	60	Etc.
5								
10								
15								
20								
30								
40								
60								
90								
120								
150								
180								
etc								

Time (Duration) – Intensity values of storms from step curve.(for use in log-log graph)

i '(mm/hr)	*t* (min)
20	
30	
35	
40	
45	
50	
55	
60	

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
	Derived values of *i* & *t* from log-log graph of above table. $i = a/t^n$ Derived value of 'a' = Derived value of 'n' = Storm Intensity Equation $i = a/t^n$ i = ... Time of concentration: As per Kirpitch Formula $Tc = [(0.885\ L^{3)}/H]^{0.385}$ Where Tc = time of concentration, minutes L = Length of overland flow in kilometres from critical (farthest) point to the inlet of drain. H = Fall in level from critical point to the inlet of drain in metres. Whether the IDF (Intensity-Duration-Frequency) curve has been drawn –Yes/No	
2.16	Whether the provision of the land / land acquisition for the SWD pumping station/mains , SWD network, if any, has been made as per 30 years requirement and future expansion in the DPR (a) Total requirement of land for: SWD Pumping Station : ………… Hectares Laying of SWD pumping mains : ………… Hectares SWD network : ………… Hectares Total : ………… Hectares Whether land in possession with Implementing Agency : Hectares (b) Whether Govt. land is yet to be transferred to the Implementing Agency and specify time required for transfer : ……..Hectare, ……… months (c) Whether private land under acquisition and time required for acquisition: ……..Hectare, ……… months (d) Status of action initiated for transfer of Govt. land and acquisition of private land (please specify) :	

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
2.17	Whether all components of storm water drainage system such as inlets, catch pits, SWD pipelines/drains, points of confluence and natural drains with outfalls have been designed as per the CPHEEO Manual and detailed drawings have been provided in the DPR	
2.18	Give Design values and infrastructure proposals for each component(use additional sheets)	
2.19	Whether the Computer Aided Design of SWD system has been furnished in DPR. Please enclose design input files (sheets) and output files (sheets) separately	
2.20	Whether the rising main of SWD system, if any, has been designed for catchment flows with respect to time of concentration and checked for minimum velocity of 0.6 m/s and maximum velocity of 3 m/s?	
2.21	Whether node spacing while designing have been adopted as per CPHEEO Manual ?	
2.22	Whether the designs of SWD pipes/drains have been checked for minimum self-cleaning velocity of 0.6 m/s by providing proper slope	
2.23	Whether surge / water hammer analysis for rising main has been calculated and furnished in the DPR	
2.24	Whether the provision for rising main units, wherever needed, such as thrust blocks, anchor blocks, expansion joints, scour / drain valves, air/vacuum releases valves and surge protection devices has been provided in the DPR	
.25	Whether drawings to scale of L-sections of SWD drains/pipelines with all details such as ground level, crown level, invert level, depths of excavation, bedding details etc., have been furnished in DPR	
2.26	Whether the configuration of the pumps proposed in SWD/drainage pumping stations is in conformity with the general guidelines of CPHEEO Manual for conveying maximum design flood, need for standby and operational capability above high flood level (HFL)	
2.27	Whether the pipe material has been selected considering the topography, efficiency in	

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
	service, ease of laying and economy in DPR	
2.28	Whether bedding conditions for different reaches of the proposed SWD pipelines/drains have been designed in the DPR as per CPHEEO Manual with reference to soil characteristics	
	Class A Bedding: Length proposed ....Km in soils of .... Classification	
	Class B Bedding: Length proposed ....Km in soils of .... Classification	
	Class C Bedding: Length proposed ....Km in soils of .... Classification	
2.29	Whether a detailed note on performance of existing SWD/drainage network and pumping station, if any has been furnished in the DPR	
2.30	Whether SWD system has provision for flood diversion to water bodies and for enabling ground water recharge	
2.31	Whether the ULBs certificate to the effect that no municipal sewage shall be discharged into the SWD system has been provided in the DPR	
2.32	Whether Bill of Qualities (BOQ) and cost estimates of individual components of drainage system prepared as per latest SOR and copy of latest Schedule of Rates (SOR) and Pro-forma invoices have been annexed with DPR. (a) Schedule of Rates adopted (please specify the year): ------------year (b) In case the SOR adopted is old, please specify the cost index for escalation approved by State Govt. (c) Any price escalation proposed in cost estimates as notified by State Govt. (d) Whether analysis of rate has been worked out for all the items and appended with DPR (e) Whether Bill of Quantities of individual component has been furnished in DPR (f) Whether lump sum(LS) provision for any item has been proposed, please specify	
2.33	Whether detailed drawing, estimation & detailed BOQ for ancillary works such as boundary wall / fencing, approach & internal road, external electrification, buildings, site development /	

<table>
<tr><th rowspan="2">S. No</th><th rowspan="2">Description</th><td>Write ‘Yes’ or ‘No’ etc. in the column below</td></tr>
<tr><td>If Yes, give Page No./DPR volume reference. If No, reasons thereof</td></tr>
<tr><td rowspan="2"></td><td>landscaping etc. has been provided in the DPR for any SWD Pumping Station</td><td></td></tr>
<tr><td>Give the General Abstract Cost Estimate and Component-wise or package-wise Abstract Cost Estimate: (use additional sheets if required)</td><td></td></tr>
<tr><td>2.34</td><td>Whether provision for DG set has been made in the DPR to tide over interruptions in power supply, if any</td><td></td></tr>
<tr><td>2.35</td><td>If yes, whether the calculations to arrive at the capacity of the same has been mentioned in the technical reports</td><td></td></tr>
<tr><td>2.36</td><td>Whether provision for road restoration has been made as per CPWD/ State PWD/ Urban Local Body norms</td><td></td></tr>
<tr><td>2.37</td><td>Give the List of Tender Packages made for ‘notice inviting tender’ (Use additional sheets if required) . Furnish the title-wise Tender packages and their value.</td><td></td></tr>
<tr><td>2.38</td><td>Calculate service level benchmark as per MoUD. Please furnish SLB.
<table>
<tr><th>Sl. No.</th><th>Indicator</th><th>Before implementation of the project</th><th>After implementation of the project</th><th>Benchmark</th></tr>
<tr><td>1.</td><td>Coverage</td><td></td><td></td><td>100%</td></tr>
<tr><td>2.</td><td>Incidence of water logging</td><td></td><td></td><td>0 numbers</td></tr>
</table></td><td></td></tr>
<tr><td>2.40</td><td>Whether project implementation period of project has been furnished in DPR

Specify the implementation period:…………..year</td><td></td></tr>
<tr><td>2.41</td><td>Whether detailed BAR Chart and PERT/CPM network showing implementation schedule has been furnished in DPR</td><td></td></tr>
<tr><td>2.42</td><td>Whether Internal rate of return (IRR) / Economic rate of return (ERR) has been furnished in DPR</td><td></td></tr>
<tr><td>2.43</td><td>Whether traffic diversion/ control arrangements for public and workers’ safety, arising out of construction phase of storm water drainage works have been furnished in the DPR</td><td></td></tr>
<tr><td>2.44</td><td>Whether Institutional and financial status of Project Executing Agency (PEA) has been reported in DPR</td><td></td></tr>
</table>

<table>
<tr><th rowspan="2">S. No</th><th rowspan="2">Description</th><th colspan="5">Write ‘Yes’ or ‘No’ etc. in the column below</th></tr>
<tr><th colspan="5">If Yes, give Page No./DPR volume reference. If No, reasons thereof</th></tr>
<tr><td rowspan="5">2.45</td><td>Whether Operation & Maintenance cost and revenue generation details (O & M Framework – existing & proposed) has been furnished in DPR

(a) Existing tariff / cess / charges (in Rs.):
Residential --
Commercial --
Institutions --
Industries --
(b) Proposed tariff/cess/charges (in Rs.)
Residential --
Commercial --
Institutions --
Industries --</td><td colspan="5"></td></tr>
<tr><td rowspan="4">(c) Annual O & M cost (Rs. in lakhs)
(i) Existing (last 5 years)

(ii) Proposed</td><td colspan="5"></td></tr>
<tr><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td colspan="5"></td></tr>
<tr><td rowspan="4"></td><td rowspan="4">(d) Annual Revenue (Rs. in lakhs)
(i) Existing (last 5 years)

(ii) Proposed</td><td colspan="5"></td></tr>
<tr><td>1</td><td>2</td><td>3</td><td>4</td><td>5</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td colspan="5"></td></tr>
</table>

S. No	Description	Write 'Yes' or 'No' etc. in the column below **If Yes, give** Page No./DPR volume reference. If **No**, reasons thereof
2.46	Whether Environmental and social problems (if applicable) has been furnished in DPR	
2.47	Whether provision has been made @ 0.5% of the project cost in the DPR for capacity building of ULBs for further O&M of the scheme after taking over the scheme from implementing agency. Please furnish the action plan for conducting capacity building programme. The action plan must specify specific actions such as the number of officials to be deployed in the project post commissioning, their designations, qualifications and training proposed to be given.	
2.48	Whether Rehabilitation and Resettlement plan (if applicable) has been given in DPR	
2.49	Whether all the hard copies of the DPR furnished along with soft copies/	
2.50	Period of completion of the project	

Signed:
Name:
Designation:

Signed:
Name:
Designation:

APPENDIX A 4.1: EXAMPLE ON PARTIAL AREA EFFECT

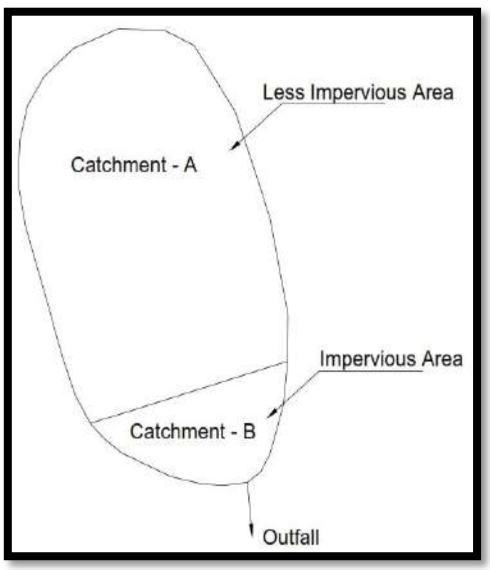
Case 1

A sub catchment has following properties:

	Catchment A	Catchment B
Run off Coefficient C	0.6	0.9
Time of concentration	30 min	5 min
Area	0.6 km^2	0.2 km^2

A storm event of 10 years Return Period having 90 Min duration rainfall results following intensities:

- **5 min – 200 mm/hr**
- **30 min – 60 mm/hr**



Find out peak runoff from the catchment for by rational method.

Solution (a)

$$Q = C_{avg} I\ A$$

Weighted average C $= \frac{C_1A_1+C_2A_2+C_3A_3+\ldots\ldots\ldots}{A_1+A_2+A_3\ldots\ldots\ldots}$

$= \frac{0.9*0.2+0.6*0.6}{0.2+0.6} = 0.675$

$$I = 60\ mm/hr$$

A+B = (0.2 + 0.6) km^2 = 0.8 km^2

K = 1/3.6

Q = (0.675*60*0.8)/3.6 = 9.0 m^3/ sec

Solution (b)

$$Q = C\ I\ A$$

C = 0.9
$I =\ 200$ mm/hr
A = 0.2 km^2
K = 1/3.6
Q = (0.9*200*0.2)/3.6 = 10.0 m^3/ sec

Maximum of the above two values shall be taken. Therefore Q = 10.0 m^3/ sec

Case II

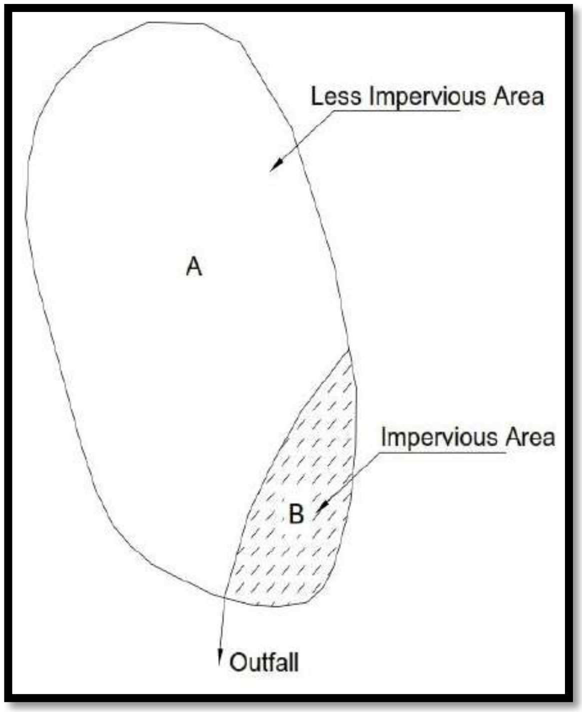
A sub catchment has following properties:

	Catchment A	**Catchment B (contributory)**
Run off Coefficient C	0.6	0.9
Time of concentration	60 min	10 min
Area	1.0 km^2	0.3 km^2

A storm event of 10 years Return Period having 90 Min duration rainfall results following intensities:

- **10 min – 130 mm/hr**
- **60 min – 40 mm/hr**

Find out peak runoff from the catchment for by rational method.



Solution (a)

Flow for Catchment A for time of concentration of 60 min

$$Q = C\ I\ A$$

K = 1/3.6

Q_1 = (0.6 × 40 × 1)/3.6 = 6.67 m^3/ sec

Flow for Catchment B for time of concentration of 60 min

Q_2 = (0.9 × 40 × 0.3)/3.6 = 3 m^3/ sec

Total Flow at Outlet = Q_1+ Q_2 = 6.67 + 3 = 9.67 m^3/ sec

Solution (b)

Flow for Catchment B for Time of concentration 10 min

$$Q = C\ I\ A$$

K = 1/3.6

Q_1 = (0.9*130*0.3)/3.6 = 9.75m^3/ sec

APPENDIX A 4.2: EXAMPLE ON TIME-AREA-METHOD

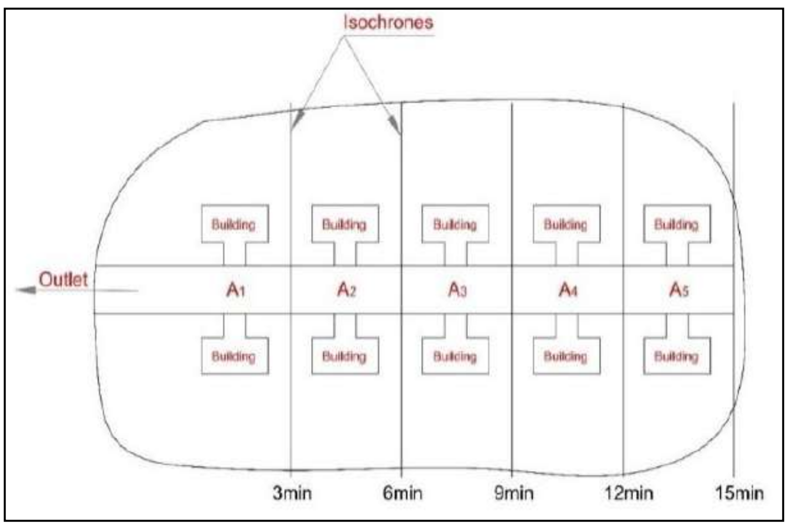
PROBLEM:

Construct the runoff hydrograph for 10 hectare catchment where total time of concentration is 15 minutes. The time distribution of rainfall and corresponding losses are given below. Use time area method to develop the hydrograph.

Time (minutes)	Rainfall depth (mm)	Infiltration and other losses (mm)	Effective rainfall(mm)
0	0	0	0
3	11.4	1.5	9.9
6	15.9	0	15.9
9	9.1	0	9.1
12	6.8	0	6.8
15	2.3	0	2.3

SOLUTION:

Draw isochrones approximately sub dividing the catchment for 3, 6, 9, 12, and 15 minutes travel time period considering total time of concentration. Measure areas between adjacent isochrones and tabulate cumulative time areas as follows.



Time (Minutes)	Cumulative area Area in m^2
0	0
3	27000
6	50000

9	69000
12	85000
15	100000

Runoff generated from each catchment due to incremental effective rainfall amount is calculated reaching the out fall.

Travel time of each zone is 3 minutes. Rainfall occurs over the entire catchment in three minutes interval as shown with losses as given in the above table taken as I_1, I_2, I_3, I_4, I_5.

Now in first 3 minutes interval I_1 rainfall has fallen over the entire catchment and therefore after 3 minutes interval the output discharge at outlet is contributed by sub-catchment A_1 from rainfall I_1. And hence, discharge $q_1 = A_1 * I_1$

Similarly I_2 rainfall has fallen in second 3 minute interval, the discharge A_2*I_1 and A_1*I_2 reach simultaneously at the outlet, $q_2= A_2 * I_1 +A_1 * I_2$

Similarly by lagging and adding $q_3 = A_3 *I_1 + A_2 *I_2 + A_1 * I_3$

$$q_4 = A_4 *I_1 + A_3 *I_2 + A_2 * I_3 + A_1 * I_4$$
$$q_5 = A_5*I_1 + A_4 * I_2 + A_3 * I_3 + A_2 * I_4 + A_1 * I_5$$

After lapse of 15 minutes the rain stops and rainfall generated by I_1 is entirely drained out at the outlet.
Rest of the incremental rainfalls falling over the sub catchment subsequently reach the outlet point as given by lagging and adding sub catchments flows here under.

$$q_6 = A_5*I_2 + A_4*I_3 + A_3*I_4 + A_2*I_5$$
$$q_7= A_5*I_3 + A_4*I_4 + A_3*I_5$$
$$q_8= A_5*I_4+A_4*I_5$$
$$q_9= A_5 * I_5$$
$$q_{10} = 0$$

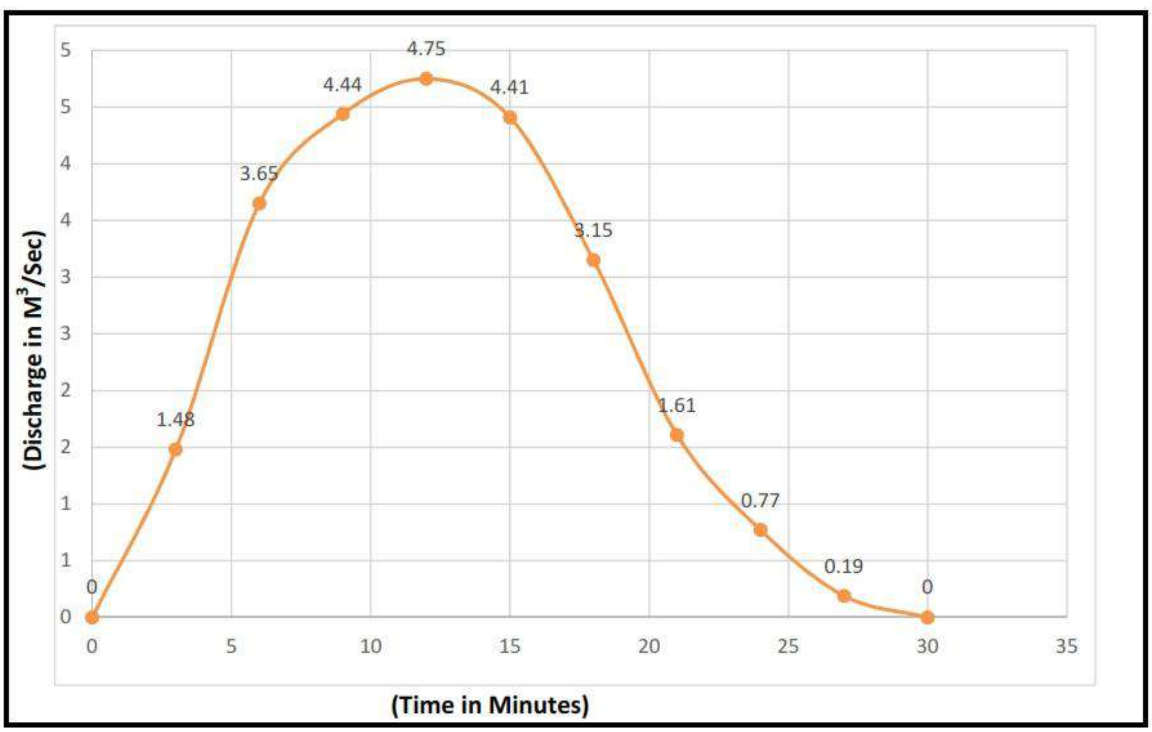
For computing if q in m^3/s, A, area between isochrones in m^2, incremental rainfall, I, in mm and time interval, t, in minutes then,

$$q = 0.001 *I*A/ 60*t$$

The total discharges after each successive interval are shown in the following table:

Time in minutes	Effective Rainfall in mm	Area of sub catchment in m^2	Runoff generated by effective rainfall in m^3/sec					Hydrograph In m^3/s
			Effective Rainfall (mm)	Effective Rainfall (mm)	Effective Rainfall (mm)	Effective Rainfall (mm)	Effective Rainfall (mm)	
0	0	0	0	0	0	0	0	0
3	9.9	27000	1.48	0				1.48
6	15.9	23000	1.26	2.39	0			3.65
9	9.1	19000	1.04	2.03	1.37	0		4.44
12	6.8	16000	0.88	1.68	1.16	1.02	0	4.75
15	2.3	15000	0.82	1.42	0.96	0.87	0.34	4.41
18	0	0	0	1.33	0.81	0.72	0.29	3.15
21	0	0	0	0	0.76	0.61	0.24	1.61
24	0	0	0	0	0	0.57	0.20	0.77
27	0	0	0	0	0	0	0.19	0.19
30	0	0	0	0	0	0	0	0

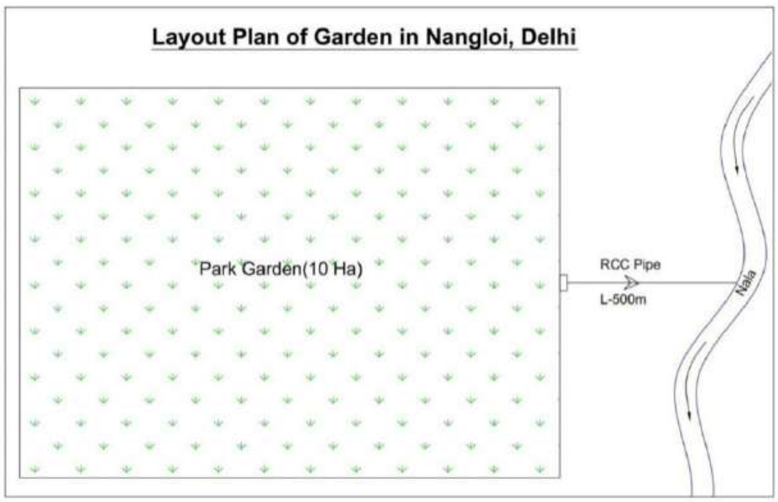
Hydrograph is drawn as shown below.



APPENDIX A 4.3: EXAMPLE ON UNIT HYDROGRAPH METHOD

PROBLEM:

A park garden in Nagloi area of Delhi city covering an area of 10 hectare drains at a single outlet as shown in the figure given below. It is proposed to drain out the storm water from the park from its outlet point to the nearest big Nallah by laying RCC pipe approximately 500.0 m in length. Design the size of pipe and determine the peak flow at the outlet of the catchment.



SOLUTION:

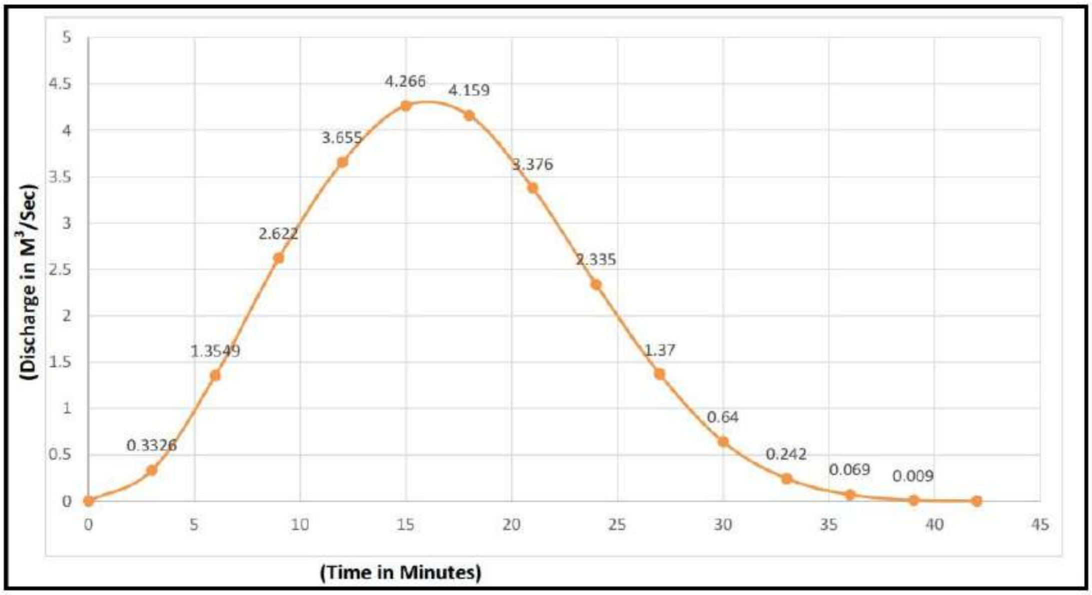
Given, the ordinates of unit hydrograph of the catchment and design hyetograph of the effective rain fall.

Unit hydrograph ordinates										
Time (min)	0	3	6	9	12	15	18	21	24	27	30
Discharge in m^3/sec	0	0.336	0.829	1.009	1.079	1.002	0.715	0.366	0.175	0.043	0

Design Hyetograph of effective rainfall	
Time in minutes	**Rainfall in cm**
0	0
3	0.99
6	1.59
9	0.91
12	0.68
15	0.23

Computation of DRH for the garden catchment							
Time (Min)	**Ordinates of UH in m^3/s**	**DRH for 0.99 cm ERH in m^3/s**	**DRH for 1.59 cm ERH in m^3/s**	**DRH for 0.91cm ERH in m^3/s**	**DRH for 0.68cm ERH in m^3/s**	**DRH for 0.23 cm ERH in m^3/s**	**DRH of the catchment (Col. 3+4+5+6+7) in m3/s**
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
0	0	0	0	0	0	0	0
3	0.336	0.3326	0	0	0	0	0.3326
6	0.829	0.8207	0.5342	0	0	0	1.3549
9	1.009	0.9989	1.3181	0.3057	0	0	2.622
12	1.079	1.0682	1.6043	0.7543	0.2284	0	3.655
15	1.002	0.9919	1.7156	0.9181	0.5637	0.6772	4.266
18	0.715	0.7078	1.5931	0.9818	0.6861	0.1906	4.159
21	0.366	0.3623	1.1368	0.9118	0.7337	0.232	3.376
24	0.175	0.1732	0.5819	0.6506	0.6813	0.2481	2.335
27	0.043	0.0425	0.2782	0.333	0.4862	0.2304	1.37
30	0	0	0.0683	0.1592	0.2488	0.1644	0.64
33	0	0	0	0.0391	0.119	0.0841	0.242
36	0	0	0	0	0.0292	0.0402	0.069
39	0	0	0	0	0	0.0098	0.009
42	0	0	0	0	0	0	0

Basic principles of theory and application of unit hydrograph to generate DRH may be referred in chapter 4.In accordance with theory of linear response that is if the rainfall excess in a duration 'D' hour is 'r' times the unit depth (1 cm), the ordinate of the resulting hydrograph will be 'r' times the corresponding ordinate of 'D' hour unit hydrograph. The table shows the ERH depth of subsequent interval multiplied by the unit hydrograph ordinates in column 2 by subsequently lagging as per time interval which is evident from the above table of computation. DRH of the park – garden is drawn and given below:



Peak flow at the outlet of the garden (from above hydrograph) = 4.266m^3/sec

Using Manning formula and taking value of 'n' 0.013 it is computed that pipe of diameter 2000 mm having a slope of 1 in 1220 shall have following hydraulic characteristics:

Q (full) = 4.35 m^3/sec
V (full) = 1.38m/sec
Q (design) =4.266 m^3/sec
V (design) = 1.58 m/sec

Therefore the above design of RCC pipe is adopted for conveying the peak flow of storm water to the receiving water of the big Nallah.

Computation:

$$\boldsymbol{Q_f = \frac{A^{5/3} * S^{1/2}}{n * p^{2/3}}}$$

Where, Q_f : Full section flow in the conduit

A : Cross section of the conduit

S : Bed slope

Substituting the values of pipe and slope parameters as given above

$$Q_f = \frac{3.14^{1.66} * (\frac{1}{1220})^{1/2}}{0.013 * 6.26^{2/3}}$$

$$= 4.354 \text{ m}^3\text{/sec}$$

V_f = 4.354/3.14 = 1.38 m/sec

From Table 5.5 of Chapter 5

Q (design)/ Q_f = 0.97, then, v(design)/ V_f =1.14, d(design)/D(full)= 0.8

Hence v (design) = 1.38*1.14 =1.58 m/sec approximately

Depth of flow = 0.8*2 = 1.60 m

APPENDIX A 5.1: EXAMPLE ON CRITICAL DEPTH

PROBLEM:
Calculate the critical depth and the corresponding specific energy for a discharge of 5.0 m^3/sec in the following channels:

a) Rectangular Channel B = 2.0 m
b) Triangular Channel m = 0.5
c) Trapezoidal Channel B = 2.0 m; m = 1.5
d) Circular Channel D = 2.0 m

Solution:

Rectangular Channel:

$q = \frac{Q}{B} = \frac{5.0}{2.0} = 2.5$ m^3/s/m

$$y_c = \sqrt[3]{\frac{q^2}{2g}} = \sqrt[3]{\frac{2.5^2}{2 \times 9.81}} = 0.860\, m$$

$\frac{E_c}{Y_c} = 1.5$; $E_c = 1.290\, m$

For Triangular Channel,

$$y_c = \sqrt[5]{\frac{2Q^2}{gm^2}} = \sqrt[5]{\frac{2 \times 5^2}{9.81 \times 0.5^2}} = 1.828\, m$$

$\frac{E_c}{Y_c} = 1.25$; $E_c = 2.284\, m$

For Trapezoidal Channel,

$$\Psi = \frac{Qm^{3/2}}{\sqrt{g}B^{5/2}} = \frac{0.5 \times 1.5^{3/2}}{\sqrt{9.81}\, 2^{5/2}} = 0.51843$$

Using Appendix 5.4, corresponding values:

ξ = $\frac{my_c}{B} = 0.536$

$Y_c = 0.715\, m$

$A_c = (2.0 + 1.5 \times 0.715) \times 0.715 = 2.197$ m^2

$V_c = \frac{5}{2.197} = 2.276$m/sec

$$E_c = y_c + \frac{v_c^2}{2g} = 0.715 + 0.264 = 0.979\, m$$

Circular Section

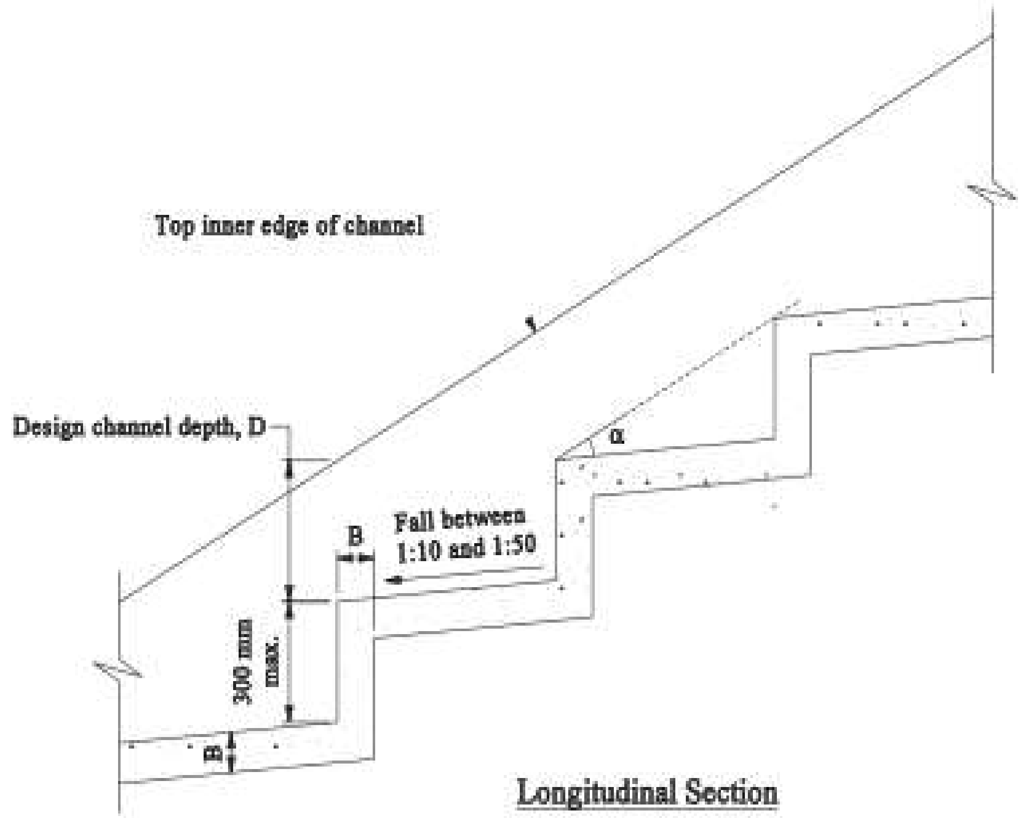
$$Z = \frac{Q}{\sqrt{g}} = \frac{5}{\sqrt{9.81}} = 1.5964$$

$$\frac{Z}{d_0^{2.5}} = \frac{1.5964}{2^{2.5}} = 0.2822$$

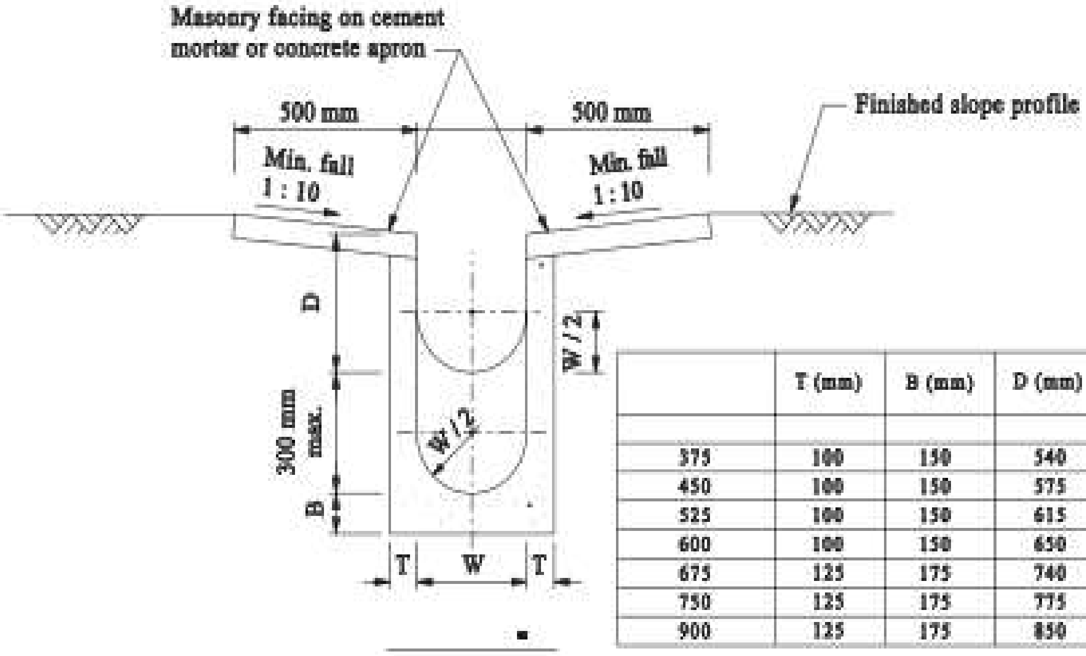
Computing value from Appendix 5.4: $\frac{y}{d_o} = 0.537$

$$y_c = 1.074\ m$$

APPENDIX A 5.2: DETAILS OF STANDARD SIZED STEPPED CHANNELS

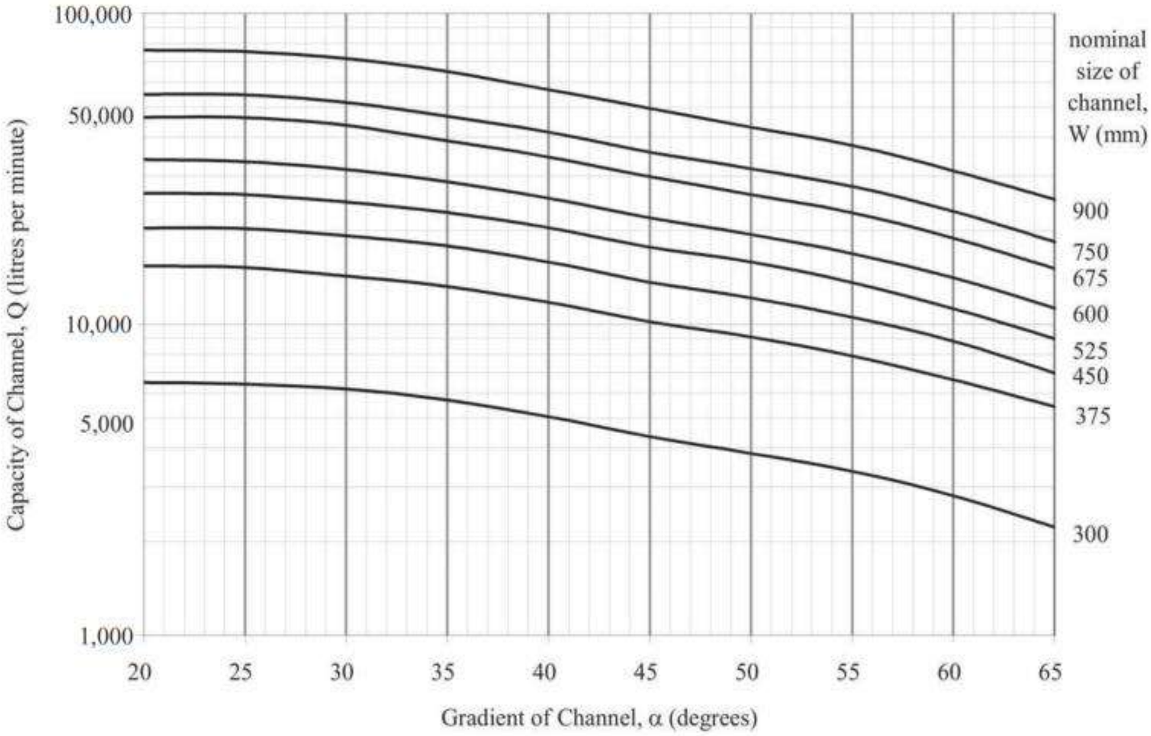


Longitudinal Section



	T (mm)	B (mm)	D (mm)
375	100	150	540
450	100	150	575
525	100	150	615
600	100	150	650
675	125	175	740
750	125	175	775
900	125	175	850

DESIGN CHART FOR STANDARD SIZED STEPPED CHANNELS



Recommended Minimum Length of Standard Sized Stepped Channels (metres)

Nominal size of channel, W (mm)	Channel gradient, α (degrees)									
	20	25	30	35	40	45	50	55	60	65
300	3.3	3.2	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0
375	5.1	5.0	4.8	4.7	4.5	4.3	4.3	4.3	4.3	4.3
450	5.5	5.4	5.2	5.1	4.8	4.6	4.6	4.6	4.6	4.6
525	6.0	5.8	5.7	5.4	5.2	5.0	5.0	5.0	5.0	5.0
600	6.4	6.3	6.1	5.8	5.6	5.3	5.3	5.3	5.3	5.3
675	7.3	7.2	6.9	6.6	6.3	6.1	6.0	6.0	6.0	6.0
750	7.8	7.6	7.3	7.0	6.7	6.4	6.4	6.3	6.3	6.3
900	8.6	8.4	8.1	7.8	7.4	7.0	7.0	7.0	7.0	7.0

APPENDIX A 5.3 : GEOMETRIC ELEMENTS FOR CIRCULAR CHANNEL SECTIONS

d_o=diameter	R=hydraulic radius
y=depth of flow	T=top width
A=water area	D=hydraulic depth
P=wetter perimeter	Z=A$\sqrt{D}$=section factor for critical-flow computation

y/d_o	A/d_o^2	P/d_o	R/d_o	T/d_o	D/d_o	$Z/d_o^{2.5}$	$AR^{2/3}/d_o^{2.5}$
0.01	0.0013	0.2003	0.0066	0.1990	0.0066	0.0001	0.0000
0.02	0.0037	0.2838	0.0132	0.2800	0.0134	0.0004	0.0002
0.03	0.0069	0.3482	0.0197	0.3412	0.0202	0.0010	0.0005
0.04	0.0105	0.4027	0.0262	0.3919	0.0268	0.0017	0.0009
0.05	0.0147	0.4510	0.0326	0.4359	0.0336	0.0027	0.0015
0.06	0.0192	0.4949	0.0389	0.4750	0.0406	0.0039	0.0022
0.07	0.0242	0.5355	0.0451	0.5103	0.0474	0.0053	0.0031
0.08	0.0294	0.5735	0.0513	0.5426	0.0542	0.0069	0.0040
0.09	0.0350	0.6094	0.0574	0.5724	0.0612	0.0087	0.0052
0.10	0.0409	0.6435	0.0635	0.6000	0.0682	0.0107	0.0065
0.11	0.0470	0.6761	0.0695	0.6258	0.0752	0.0129	0.0079
0.12	0.0534	0.7075	0.0754	0.6499	0.0822	0.0153	0.0095
0.13	0.0600	0.7377	0.0813	0.6728	0.0892	0.0179	0.0113
0.14	0.0668	0.7670	0.0871	0.6940	0.0964	0.0217	0.0131
0.15	0.0739	0.7954	0.0929	0.7141	0.1034	0.0238	0.0152
0.16	0.0811	0.8230	0.0986	0.7332	0.1106	0.0270	0.0173
0.17	0.0885	0.8500	0.1042	0.7513	0.1178	0.0304	0.0196
0.18	0.0961	0.8763	0.1042	07684	0.1252	0.0339	0.0220
0.19	0.1039	0.9020	0.1152	0.7846	0.1324	0.0378	0.0247
0.20	0.1118	0.9273	0.1206	0.8000	0.1398	0.0418	0.0273
0.21	0.1190	0.9521	0.1259	0.8146	0.1472	0.0460	0.0301
0.22	0.1281	0.9764	0.1312	0.8285	0.1546	0.0503	0.0333
0.23	0.1365	1.0003	0.1364	0.8417	0.1662	0.0549	0.0359
0.24	0.1449	1.0239	0.1416	0.8542	0.1696	0.0597	0.0394
0.25	0.1535	1.0472	0.1466	0.8660	0.1774	0.0646	0.0427
0.26	0.1623	1.0701	0.1516	0.8773	0.1850	0.0697	0.0464
0.27	0.1711	1.0928	0.1566	0.8879	0.1926	0.0751	0.0497
0.28	0.1800	1.1152	0.1614	0.8980	0.2004	0.0805	0.0536
0.29	0.1890	1.1373	0.1662	0.9075	0.2084	0.0862	0.0571
0.30	0.1982	1.1593	0.1709	0.9165	0.2162	0.0921	0.0610
0.31	0.2074	1.1810	0.1755	0.9250	0.2242	0.0981	0.0650
0.32	0.2167	1.2025	0.1801	0.9330	0.2322	0.1044	0.0690
0.33	0.2260	1.2239	0.1848	0.9404	0.2404	0.1107	0.0736
0.34	0.2355	1.2451	0.1891	0.9474	0.2486	0.1172	0.0776
0.35	0.2450	1.2661	0.1935	0.9539	0.2568	0.1241	0.0820
0.36	0.2546	1.2870	0.1978	0.9600	0.2652	0.1310	0.0864
0.37	0.2642	1.3078	0.2020	0.9656	0.2736	0.1381	0.0909
0.38	0.2739	1.3284	0.2061	0.9708	0.2822	0.1453	0.0955

0.39	0.2836	1.3490	0.2102	0.9755	0.2908	0.1528	0.1020
0.40	0.2934	1.3694	0.2142	0.9798	0.2994	0.1603	0.1050
0.41	0.3032	1.3898	0.2181	0.9837	0.3082	0.1682	0.1100
0.42	0.3132	1.4101	0.2220	0.9871	0.3172	0.1761	0.1147
0.43	0.3229	1.4303	0.2257	0.9902	0.3262	0.1844	0.1196
0.44	0.3328	1.4505	0.2294	0.9928	0.3352	0.1927	0.1245
0.45	0.3428	1.4706	0.2331	0.9950	0.3446	0.2011	0.1298
0.46	0.3527	1.4907	0.2366	0.9968	0.3538	0.2098	0.1348
0.47	0.3627	1.5108	0.2400	0.9982	0.3634	0.2186	0.1401
0.48	0.3727	1.5308	0.2434	0.9992	0.3730	0.2275	0.1452
0.49	0.3827	1.5508	0.2467	0.9998	0.3828	0.2366	0.1505
0.50	0.3927	1.5708	0.2500	1.0000	0.3928	0.2459	0.1558
0.51	0.4027	1.5908	0.2531	0.9998	0.4028	0.2553	0.1610
0.52	0.4127	1.6108	0.2561	0.9992	0.4130	0.2650	0.1664
0.53	0.4227	1.6308	0.2591	0.9982	0.4234	0.2748	0.1715
0.54	0.4327	1.6509	0.2620	0.9968	0.4340	0.2848	0.1772
0.55	0.4426	1.6710	0.2649	0.9950	0.4448	0.2949	0.1825
0.56	0.4526	1.6911	0.2676	0.9928	0.4558	0.3051	0.1878
0.57	0.4625	1.7113	0.2703	0.9902	0.4670	0.3158	0.1933
0.58	0.4723	1.7315	0.2728	0.9871	0.4786	0.3263	0.1987
0.59	0.4822	1.7518	0.2753	0.9887	0.4902	0.3373	0.2041
0.60	0.4920	1.7722	0.2776	0.9798	0.5022	0.3484	0.2092
0.61	0.5018	1.7926	0.2797	0.9755	0.5144	0.3560	0.2146
0.62	0.5115	1.8132	0.2818	0.9708	0.5270	0.3710	0.2199
0.63	0.5212	1.8338	0.2839	0.9656	0.5398	0.3830	0.2252
0.64	0.5308	1.8546	0.2860	0.9600	0.5530	0.3945	0.230.
0.65	0.5404	1.8755	0.2881	0.9539	0.5666	0.4066	0.2358
0.66	0.5499	1.8965	0.2899	0.9474	0.5804	0.4188	0.2407
0.67	0.5594	1.9177	0.2917	0.9404	0.5948	0.4309	0.2460
0.68	0.5687	1.9391	0.2935	0.9330	0.6096	0.4437	0.2510
0.69	0.5780	1.9606	0.2950	0.9250	0.6250	0.4566	0.2560
0.70	0.5872	1.9823	0.2962	0.9165	0.6408	0.4694	0.2608
0.71	0.5964	2.0042	0.2973	0.9075	0.6572	0.4831	0.2653
0.72	0.6054	2.0264	0.2984	0.8980	0.6742	0.4964	0.2702
0.73	0.6143	2.0488	0.2995	0.8879	0.6918	0.5100	0.2751
0.74	0.6231	2.0714	0.3006	0.8773	0.7104	0.5248	0.2794
0.75	0.6318	2.0944	0.3017	0.8660	0.7296	0.5392	0.2840
0.76	0.6404	2.1176	0.3025	0.8542	0.7498	0.5540	0.2888
0.77	0.6489	2.1412	0.3032	0.8417	0.7710	0.5695	0.2930
0.78	0.6573	2.1652	0.3037	0.8285	0.7394	0.5850	0.2969
0.79	0.6655	2.1895	0.3040	0.8146	0.8170	0.6011	0.3008
0.80	0.6736	2.2143	0.3042	0.8000	0.8420	0.6177	0.3045
0.81	0.6815	2.2395	0.3044	0.7846	0.8686	0.6347	0.3082
0.82	0.6893	2.2653	0.3043	0.7684	0.8970	0.6524	0.3118
0.83	0.6969	2.2916	0.3041	0.7513	0.9276	0.6707	0.3151
0.84	0.7043	2.3186	0.3038	0.7332	0.9606	0.6897	0.3182
0.85	0.7115	2.3462	0.3033	0.7141	0.9964	0.7098	0.3212
0.86	0.7186	2.3746	0.3026	0.6940	1.0354	0.7307	0.3240

0.87	0.7254	2.4038	0.3017	0.6726	1.0784	0.7528	0.3264
0.88	0.7320	2.4341	0.3008	0.6499	1.1264	0.7754	0.3286
0.89	0.7380	2.4655	0.2996	0.6258	1.1800	0.8016	0.3307
0.90	0.7445	2.4981	0.2980	0.6000	1.2408	0.8285	0.3324
0.91	0.7504	2.5322	0.2963	0.5724	1.3110	0.8586	0.3336
0.92	0.7560	2.5681	0.2944	0.5426	1.3932	0.8917	0.3345
0.93	0.7612	2.6061	0.2922	0.5103	1.4918	0.9292	0.3350
0.94	0.7662	2.6467	0.2896	0.4750	1.6130	0.9725	0.3353
0.95	0.7707	2.6906	0.2864	0.4359	1.7682	1.0242	0.3349
0.96	0.7749	2.7389	0.2830	0.3919	1.9770	1.0888	0.3340
0.97	0.7785	2.7934	0.2787	0.3412	2.2820	1.1752	0.3322
0.98	0.7816	2.8578	0.2735	0.2800	2.7916	1.3050	0.3291
0.99	0.7841	2.9412	0.2665	0.1990	3.9400	1.5554	0.3248
1.00	0.7854	3.1416	0.2500	0.0000	~~	~~	0.3117

APPENDIX A 5.4: VALUES FOR COMPUTATION OF CRITICAL DEPTH IN TRAPEZOIDAL CHANNEL

ξ	Ψ	ξ	Ψ	ξ	Ψ	ξ	Ψ	ξ	Ψ
0.100	0.0333042	0.330	0.2256807	0.560	0.5607910	0.790	1.0469124	1.020	1.6962526
0.105	0.0359281	0.335	0.2314360	0.565	0.5697107	0.795	1.0592476	1.025	1.7122746
0.110	0.0386272	0.340	0.2372580	0.570	0.5787019	0.800	1.0716601	1.030	1.7283798
0.115	0.0414006	0.345	0.2431469	0.575	0.5877645	0.805	1.0841500	1.035	1.7445682
0.120	0.0442474	0.350	0.2491026	0.580	0.5968989	0.810	1.0967174	1.040	1.7608400
0.125	0.0471671	0.355	0.2551252	0.585	0.6061050	0.815	1.1093625	1.045	1.7771953
0.130	0.0501588	0.360	0.2612149	0.590	0.6153829	0.820	1.1220854	1.050	1.7936343
0.135	0.0532222	0.365	0.2673716	0.595	0.6247330	0.825	1.1348861	1.055	1.8101570
0.140	0.0563565	0.370	0.2735954	0.600	0.6341551	0.830	1.1477649	1.060	1.8267635
0.145	0.0595615	0.375	0.2798865	0.605	0.6436496	0.835	1.1607219	1.065	1.8434541
0.150	0.0628365	0.380	0.2862449	0.610	0.6532164	0.840	1.1737572	1.070	1.8602288
0.155	0.0661812	0.385	0.2926706	0.615	0.6628558	0.845	1.1868709	1.075	1.8770877
0.160	0.0695953	0.390	0.2991638	0.620	0.6725678	0.850	1.2000631	1.080	1.8940310
0.165	0.0730784	0.395	0.3057246	0.625	0.6823525	0.855	1.2133341	1.085	1.9110589
0.170	0.0766302	0.400	0.3123531	0.630	0.6922102	0.860	1.2266838	1.090	1.9281713
0.175	0.08022504	0.405	0.3190493	0.635	0.7021409	0.865	1.2401125	1.095	1.9453685
0.180	0.0839387	0.410	0.3258133	0.640	0.7121448	0.870	1.2536203	1.100	1.9626506
0.185	0.0876950	0.415	0.3326452	0.645	0.7222220	0.875	1.2672072	1.105	1.9800176
0.190	0.0915190	0.420	0.3395452	0.650	0.7323725	0.880	1.2808735	1.110	1.9974698
0.195	0.0954105	0.425	0.3465132	0.655	0.7425966	0.885	1.2946192	1.115	2.0150072
0.200	0.0993694	0.430	0.3535495	0.660	0.7528944	0.890	1.3084445	1.120	2.0326299
0.205	0.1033955	0.435	0.3606541	0.665	0.7632659	0.895	1.3223496	1.125	2.0503382
0.210	0.1074887	0.440	0.3678272	0.670	0.7737114	0.900	1.3363344	1.130	2.0681321
0.215	0.1116488	0.445	0.3750688	0.675	0.7842309	0.905	1.3503992	1.135	2.0860117
0.220	0.1158757	0.450	0.3823789	0.680	0.7948246	0.910	1.3645441	1.140	2.1039771
0.225	0.1201694	0.455	0.3897579	0.685	0.8054926	0.915	1.3787693	1.145	2.1220286
0.230	0.1245297	0.460	0.3972056	0.690	0.8162350	0.920	1.39330747	1.150	2.1401661
0.235	0.1289566	0.465	0.4047224	0.695	0.8270520	0.925	1.4074607	1.155	2.1583899
0.240	0.1334500	0.470	0.4123082	0.700	0.8379437	0.930	1.4219272	1.160	2.1767000
0.245	0.13890098	0.475	0.4199631	0.705	0.8489102	0.935	1.4364745	1.165	2.1950965
0.250	0.1426361	0.480	0.4276873	0.710	0.8599516	0.940	1.4511026	1.170	2.2135797
0.255	0.1473287	0.485	0.4354810	0.715	0.8710681	0.945	1.4658118	1.175	2.2321496
0.260	0.1520877	0.490	0.4433441	0.720	0.882598	0.950	1.4806020	1.180	2.2508063
0.265	0.1569130	0.495	0.4512768	0.725	0.8935269	0.955	1.4954734	1.185	2.2695499
0.270	0.1618046	0.500	0.4592793	0.730	0.9048694	0.960	1.5104263	1.190	2.2883806
0.275	0.1667625	0.505	0.4673517	0.735	0.9162875	0.965	1.5254606	1.195	2.3072986
0.280	0.1717868	0.510	0.4754940	0.740	0.9277813	0.970	1.5405765	1.200	2.3263038
0.285	0.1768773	0.515	0.4837063	0.745	0.9393510	0.975	1.5557742	1.205	2.3453965
0.290	0.1820342	0.520	0.4919889	0.750	0.9509966	0.980	1.5710537	1.210	2.3645767
0.295	0.172575	0.525	0.5003418	0.755	0.9627183	0.985	1.5864153	1.215	2.3838447
0.300	0.1925471	0.530	0.5087651	0.760	0.9745163	0.990	1.6018590	1.220	2.4032004
0.305	0.1979031	0.535	0.5172590	0.765	0.9863907	0.995	1.6173849	1.225	2.4226440
0.310	0.2033256	0.540	0.5258236	0.770	0.9983415	1.000	1.6329932	1.230	2.4421757
0.315	0.2088145	0.545	0.5344589	0.775	1.0103690	1.005	1.6486840	1.235	2.4617956

0.320	0.2143700	0.550	0.5431652	0.780	1.0224732	1.010	1.6644574	1.240	2.4815037
0.325	0.2199920	0.555	0.5519425	0.785	1.0346543	1.015	1.6803135	1.245	2.5013003
0.330	0.2256807	0.560	0.5607910	0.790	1.0469124	1.020	1.6962526	1.250	2.5211853

Where,

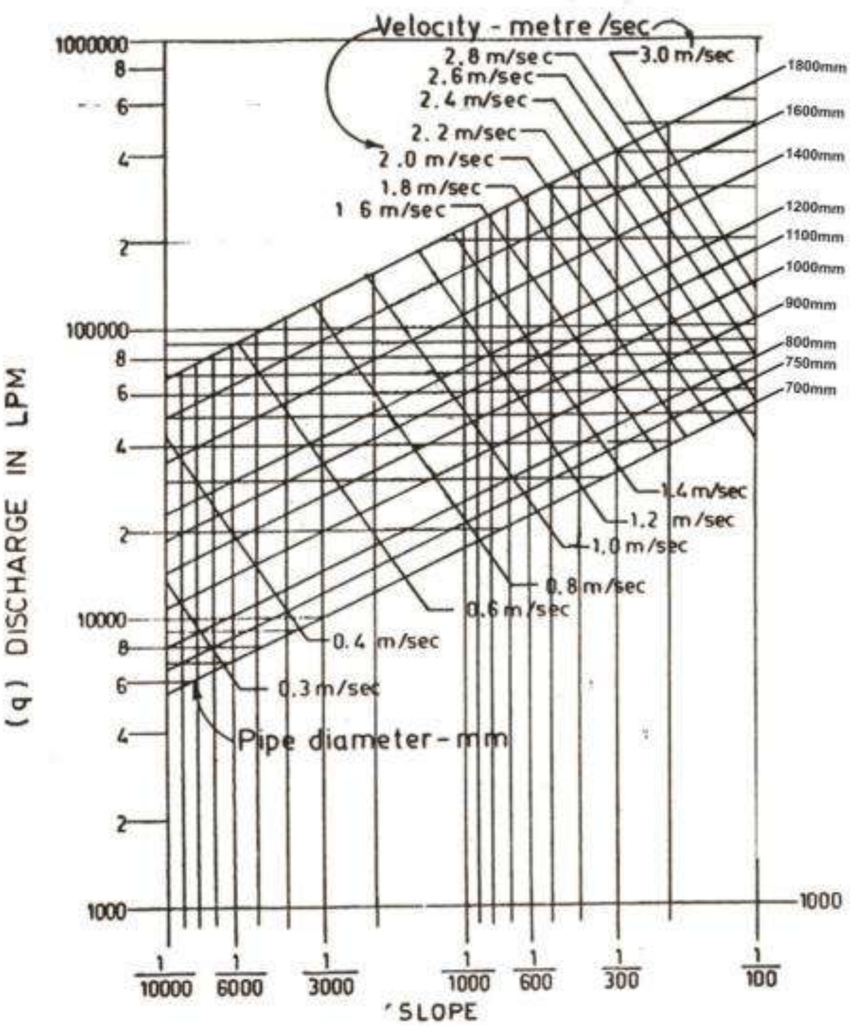
$$\Psi = \frac{Qm^{\frac{3}{2}}}{\sqrt{g}B^{\frac{5}{2}}} \quad \text{And,} \quad \xi = \frac{my_c}{B}$$

APPENDIX A 5.5 (A): NOMOGRAM FOR MANNING'S FORMULA

NOMOGRAM FOR MANNING'S FORMULA FOR GRAVITY SEWERS FLOWING FULL AND MANNING'S N VALUE OF 0.013
(For discharges from 1000 lpm to 1000000 lpm)

For other values of Manning's n the velocity and discharge will be inversely proportional.
Example-Find the discharge and velocity of a sewer flowing full of diameter 900 mm, slope of 1 in 1,000 and a Manning's n value of 0.012.

Answer-From the nomogram, V = 0.90 m/s and discharge = 35,000 lpm. For n value of 0.0125, V = 0.90 × 0.013/0.0125 = 0.94 m/s & discharge = 35,000 × 0.013/0.0125 = 36,400 lpm



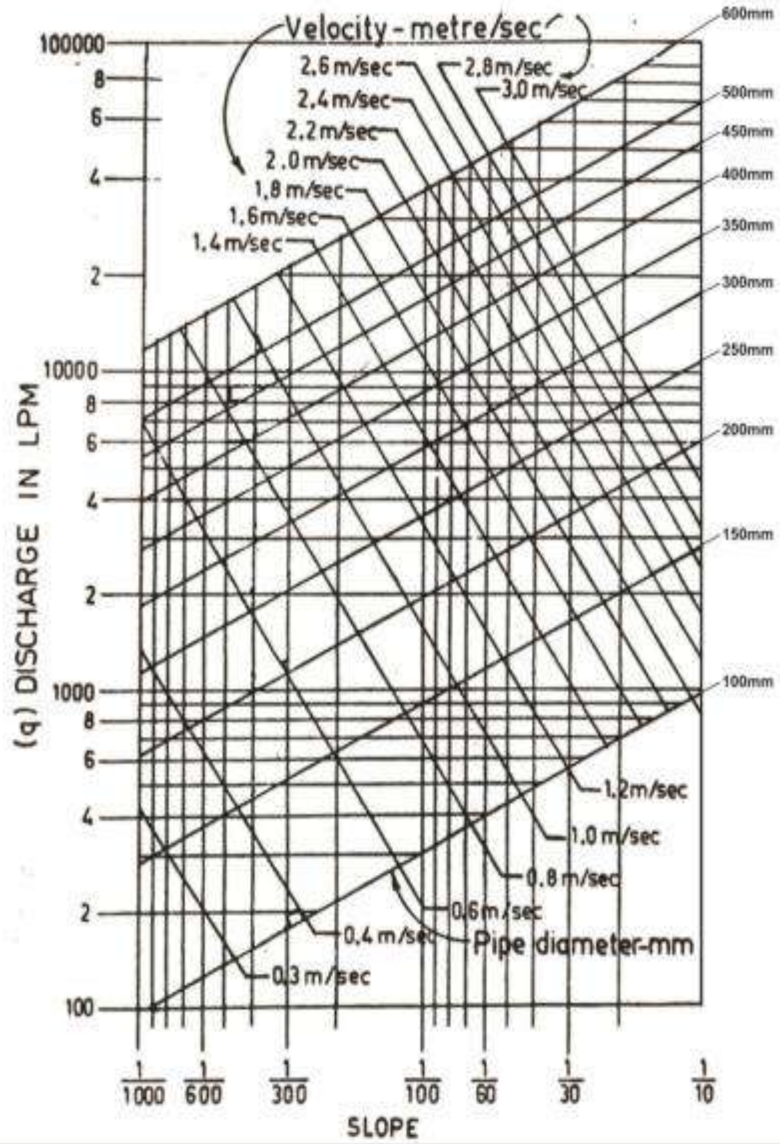
Source-"Manual on Sewerage and Sewage Treatment Systems", CPHEEO, 2013

APPENDIX A 5.5 (B): NOMOGRAM FOR MANNING'S FORMULA

NOMOGRAM FOR MANNING'S FORMULA FOR GRAVITY SEWERS FLOWING FULL AND MANNING'S N VALUE OF 0.013.
(For discharges from 100 lpm to 100000 lpm)

For other values of Manning's n, the velocity and discharge will be inversely proportional.
Example-Find the discharge and velocity of a sewer flowing full of diameter 200 mm, slope of 1 in 200 and a Manning's n value of 0.0125.

Answer-From the nomogram, V = 0.75 m/s and discharge = 1,300 lpm. For n value of 0.0125,
V = 0.75 × 0.013/0.0125 = 0.78 m/s & discharge = 1,300 × 0.013/0.0125 = 1,352 lpm



Source-"Sewerage Manual", CPHEEO, 2013

APPENDIX A 5.5 (C): NOMOGRAM FOR HAZEN WILLIAMS FORMULA

NOMOGRAM FOR HAZEN WILLIAMS FORMULA FOR MAINS

FLOWING FULL AND C VALUE OF 100

(For discharges from 100 to 100000 lpm)

For other values of C, the velocity and discharge will be directly proportional.

Example-Find the discharge and velocity of a sewer of diameter 300 mm flowing full slope of 1 in 100 and a Hazen Williams C value of 130

Answer-From the nomogram, V = 0.75 m/s and discharge = 5,700 lpm. For C value of 130, V = 0.75 × 130 / 100 = 0.98 m/s & discharge = 5,700 × 130 / 100 = 7,400 lpm

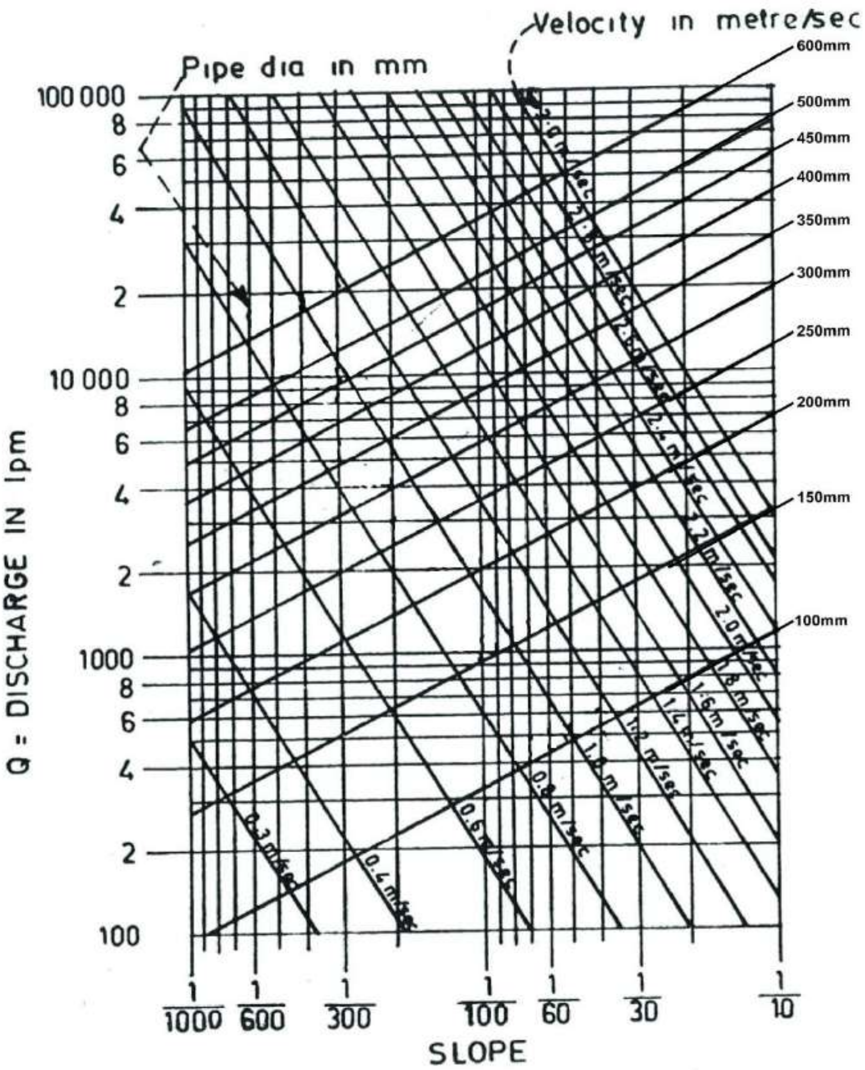


FIG 3 NOMOGRAM CHART FOR HAZEN-WILLIAMS FORMULA (C = 100) FOR Q = 100 lpm TO 100000 lpm

Source:Sewerage Manual", CPHEEO, 2013

APPENDIX A 5.5 (D): NOMOGRAM FOR HAZEN WILLIAMS FORMULA

NOMOGRAM FOR HAZEN WILLIAMS FORMULA FOR MAINS FLOWING FULL AND C VALUE OF 100 (For discharges from 1000 to 1000000 lpm)

For other values of C the velocity and discharge will increase pro rata.
Example-Find the discharge and velocity of a sewer flowing full of diameter 1,200 mm, slope of 1 in
1,000 and a Hazen Williams C value of 130
Answer-From the nomogram, V = 0.95 m/s and discharge = 63,000 lpm. For C value of 130,
V = 0.95 × 130 / 100 = 1.24 m/s & discharge = 63,000 × 130 / 100 = 81,900 lpm

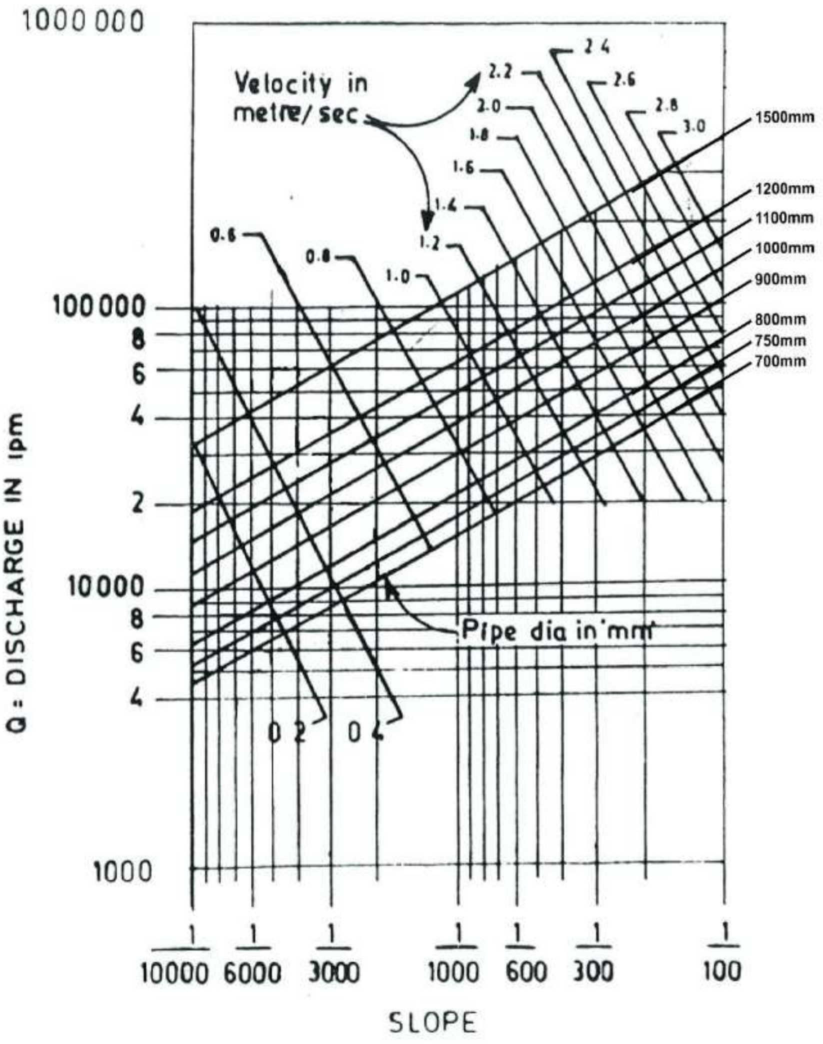
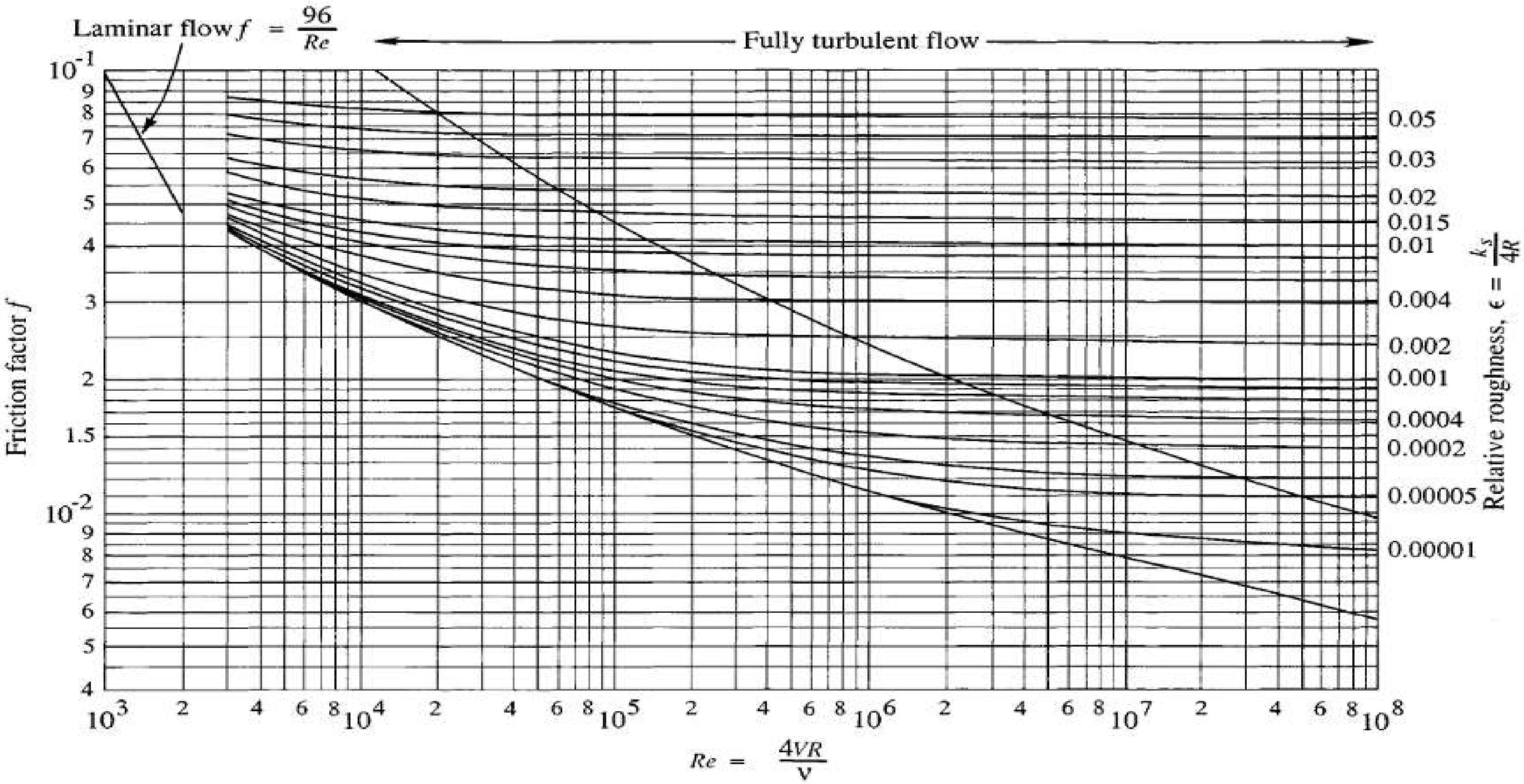


FIG 4 NOMOGRAM CHART FOR HAZEN-WILLIAMS FORMULA

(C = 100) FOR Q= 1000 lpm TO 1000000 lpm

Source: "Sewerage Manual", CPHEEO, 2013

APPENDIX A 5.6: MOODY DIAGRAM FOR OPEN CHANNEL FLOW



APPENDIX A 5.7: MANNING'S ROUGHNESS COEFFICIENT FOR OVERLAND FLOW

Source	Ground Cover	n	Range
Crawford and Linsley (1966)[a]	Smooth asphalt	0.01	
	Asphalt of concrete paving	0.014	
	Packed clay	0.03	
	Light turf	0.20	
	Dense turf	0.35	
	Dense shrubbery and forest litter	0.4	
Engman (1986)[b]	Concrete or asphalt	0.011	0.010-0.013
	Bare Sand	0.010	0.01-0.016
	Graveled surface	0.02	0.012-0.03
	Bare clay-loam (eroded0	0.02	0.012-0.033
	Range (natural)	0.13	0.01-0.32
	Bluegrass sod	0.45	0.39-0.63
	Short grass prairie	0.15	0.10-0.20
	Bermuda grass	0.41	0.30-0.48
Yen (2001)[c]	Smooth asphalt pavement	0.012	0.010-0.015
	Smooth impervious surface	0.013	0.011-0.015
	Tar and sand pavement	0.014	0.012-0.016
	Concrete pavement	0.017	0.014-0.020
	Rough impervious surface	0.019	0.015-0.023
	Smooth bare packed soil	0.021	0.017-0.025
	Moderate bare packed soil	0.030	0.025-0.035
	Rough bare packed soil	0.038	0.032-0.045
	Gravel soil	0.032	0.025-0.045
	Mowed poor grass	0.038	0.030-0.045
	Average grass, closely clipped sod	0.050	0.040-0.060
	Pasture	0.055	0.040-0.070
	Timberland	0.090	0.060-0.120
	Dense grass	0.090	0.060-0.120
	Shrubs and bushes	0.120	0.080-0.180
	Business land use	0.022	0.014-0.035
	Semi-business land use	0.035	0.022-0.050
	Industrial land use	0.035	0.020-0.050
	Dense residential land use	0.040	0.025-0.060
	Suburban residential land use	0.055	0.030-0.080
	Parks and lawns	0.075	0.040-0.120

[a]Obtained by calibration of Stanford Watershed Model.
[b]Computed by Engman (1986) by kinematic wave and storage analysis of measured rainfall-runoff data.
[c]Computed on basis of kinematic wave analysis.

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