OPSC-AEE 2020

Odisha Public Service Commission

Assistant Executive Engineer

Civil Engineering

Environmental Engineering

Well Illustrated **Theory** *with*

Solved Examples and **Practice Questions**

MADE EASY Publications

MADE EASY

Publications

Environmental Engineering

Contents

CHAPTER 6 Design of Sewerage System and Sewer Appurtenances

6.1 Introduction

Providing an adequate sewer system for an area is an art that requires careful engineering. It should be properly and skillfully, planned and designed so as to transport the entire sewage effectively and efficiently from the houses and up to the point of disposal. The sewer must be adequate in size or they will overflow and causes property damage, danger to health and nuisance. Adequacy in size of sewer calls for correct estimation of the amount of sewage and use of hydraulics to determine proper size and grades of sewers, which will permit reasonable velocity of flow. This flow should neither be too large as to require heavy excavations and high lift pumping nor should it be too small to cause deposition of solids in sewer bottom with accompanying odours and stoppages.

6.2 Difference in the Design of Water Supply Pipes and Sewer Pipes

The hydraulic design of sewers and drains, which means finding out their sections and gradients, is generally carried out on the same lines as that of the water supply pipes. There are two major differences between the characteristics of flows in sewers and water supply pipes.

(i) Water supply pipes carry water under pressure, and hence, within certain limits, they may be carried up and down the hills and the valleys; whereas, the sewer pipes carry sewage as gravity conduits and they must, therefore, be laid at a continuous gradient in the downward direction up to the outfall point, from where it will be lifted up, treated and disposed off.

(ii) Water supply pipes carry pure water without containing any kind of solid particles, either organic or inorganic in nature. The sewage, on the other hand, does contain such particles in suspension and the heavier of these particles may settle down at the bottom of the sewers as and when the flow velocity reduces, thus ultimately resulting in the clogging of the sewers. In order to avoid such clogging or silting of sewers, it is necessary that the sewer pipes be of such a size and laid at such a gradient as to generate self-cleansing velocities at different possible discharges. The sewer materials must also be capable of resisting the wear and tear caused due to abrasion of the solid particles present in the sewage, with the interior of the pipe.

6.3 Laying of Sewer Pipes

All the sewer pipes are generally laid starting from their outfall ends, towards their starting ends. The advantage gained in starting from the tail end, (i.e. outfall end) is the utilisation of the tail length even during the initial period of its construction, thus ensuring that the functioning of the sewerage scheme has not to wait till the completion of the entire scheme.

The laying of the sewer pipes is, therefore, started from the outfall end and proceeded upward by locating the different points along the proposed alignment on the ground. It is common practice, to first locate the points where manholes are required to be constructed and then laying the sewer pipe between the two manholes.

The laying of the sewer consists of the followings steps:

6.3.1 Marking of the Alignment

The alignment of the sewer is marked along the road with a theodolite and invar tape. The centre line may be marked according to the following two methods:

(a) By Reference Line
(b) By Sight Rail

(a) By Reference Line: In Reference Line method, a reference line is marked along any side of the busy roads by theodolite and invar tape. The points F_1F_2,...... are on the reference line. The starting point (P_1) of the centre line is marked with a peg. Then the distance F_1P_1 is measured by tape. Now the other points P_2, P_3, P_4 ..... etc. are marked pegs by taking as $F_1P_1 = F_2P_2 = F_3P_3$ ..... etc.

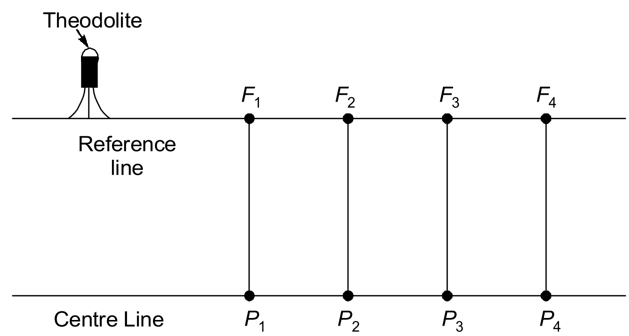


Fig. 6.1 Reference Line

Thus, the points P_1, P_2, P_3 ..... etc. will represent the centre line of the sewer. This centre line may be checked by the theodolite. (Fig. 6.1)

(b) By Sight Rail : In Sight Rail method, two vertical posts are driven at suitable distance apart. Then by ranging through a theodolite the centre line is marked with nail on a sight rail which is fixed on the vertical posts. The sight rail should be fixed in such a way so that its upper edge just coincides with the line of sight. The centre line of the sewer is transferred to the ground by plumb bob with respect to the nail.

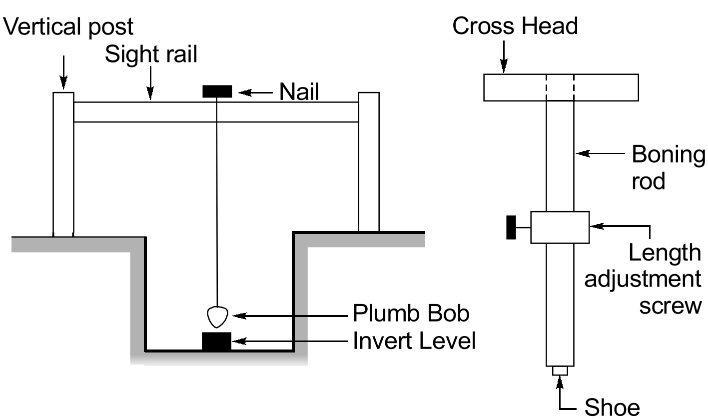


Fig. 6.2 Sight rail

The distance between the upper edge of sight rail and the invert level is determined and noted on the sight rail for finding the exact invert level by boning rod. The length of boning rod is adjusted according to the height as noted in sight rail. The crosshead is levelled with the upper edge of sight rail and the bottom edge indicates the invert level.

6.3.2 Excavation of Trench

- The width of excavation of any level will depend upon the width of the trench at the bottom and the additions due to side slopes and due to timbering etc.
- The trench is excavated between two manholes, and the sewer is laid between them.
- Further excavations are carried out for laying the pipes between the next consecutive manholes.
- The process is continued from the outfall end of the sewer towards the uphill, till the entire sewer is laid out.

6.3.3 Bracing of the Trench

- The braces are the cross wooden pieces extending from one side of the trench to the other side and may also be called struts.
- The bracing will absorb the soil pressure and prevent it from collapsing.

6.3.4 Dewatering of Trench

- While excavating a trench, the ground water may appear, if the watertable happen to be high or if the sewer happens to be laid very deep. This ground water will create problems in further excavations.
- The ground water may be removed through an open jointed drain constructed below the sewer trench, which discharges into an independent water course either by gravity or pumping.

6.3.5 Laying of Pipes

After the bedding concrete has been laid in the required alignment and levels, the sewer pipes are lowered down in to the trench. (Fig. 6.3)

NOTE

The sewer pipe lengths are usually laid from the lowest point with their socket ends facing upstream.

- The spigot of each, is inserted in the socket of the laid pipe
- After fitting the socket, spigot joining is done with lead caulking or cement mortar.

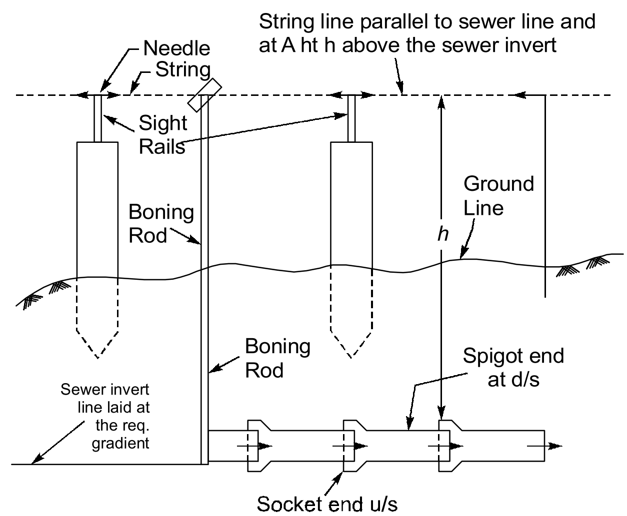


Fig. 6.3 Laying of sewer (L-section)

6.3.6 Testing of the Sewer Pipes

The sewers after being laid and jointed are tested for water tight joints, and also for correct straight alignment, as described below:

1. Test for Leakage called Water Test

The sewers are tested, so as to ensure 'no leakage' through their joints after giving a sufficient time to these joints to set in. The sewer pipe sections are tested between manhole to manhole under a test pressure of about 1.5 m of water head.

In order to carry out this test on a sewer line between two manholes, the lower end (i.e. downstream end) of the sewer is first of all, plugged. The water is now filled in the manhole at the upper end, and is allowed to flow through the sewer line. The depth of water in the manhole is maintained to the testing head about 1.5 m. The sewer line is watched by moving along the trench, and the joints which leak or sweat are repaired.

2. Test for Straightness of Alignment and Obstruction

The straightness of the sewer pipe can be tested by placing a mirror at one end of the sewer line and a lamp at other end. If the pipe line is straight, the full circle of light will be observed. However, if the pipe line is not straight, this would be apparent, and the mirror will also indicate any obstruction in the pipe barrel.

3. By Air Test

The Air Test is carried out for large diameter sewer. The pipe ends of both the manholes are plugged. An air compressor is connected to the plug the upper manhole and pressure gauge is attached with the plug of lower manhole. The pressure exerted by the compressed air is recorded in the pressure gauge. It is left for few hours. If the pressure drops below the permissible limit, then it is an indication of leakage.

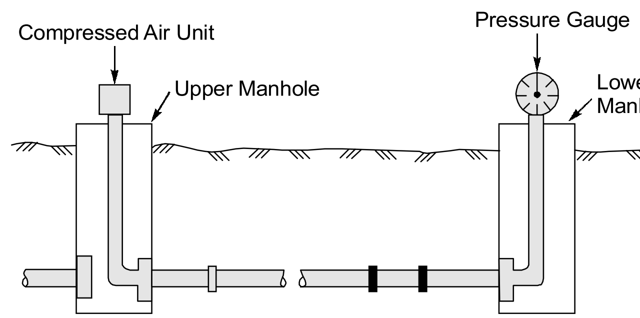


Fig. 6.4 Air Test

The exact point of leakage is found out by applying soap solution which will show bubbles at the point of leakage. If leakage is detected, it should be removed immediately.

6.4 Types of Collection System

For the transport of entire sewage effectively and efficiently from the houses and upto the point of disposal, sewer should be designed not to flow full under gravity because reserve space in the sewer safeguards against fluctuations in sewage flow.

The three types of collection systems are:

6.4.1 Sanitary Sewer

- In sanitary sewer system, lateral sewer collects discharges from houses and carry them to another branch sewer, and has no tributary sewer lines (Fig. 6.5).
- Branches or sub-main lines receive waste-water from laterals and convey it to large mains.
- The main sewer, also known as trunk or outfall sewer, carries the discharge from large areas to the treatment plant.
- Manholes are provided at intersection of sewer lines and also at regular intervals to facilitate regular inspection and cleaning.
- They are designed to carry domestic wastes originating from the sanitary conveniences of dwellings, business buildings, factories or institutions including industrial wastes produced in the area.

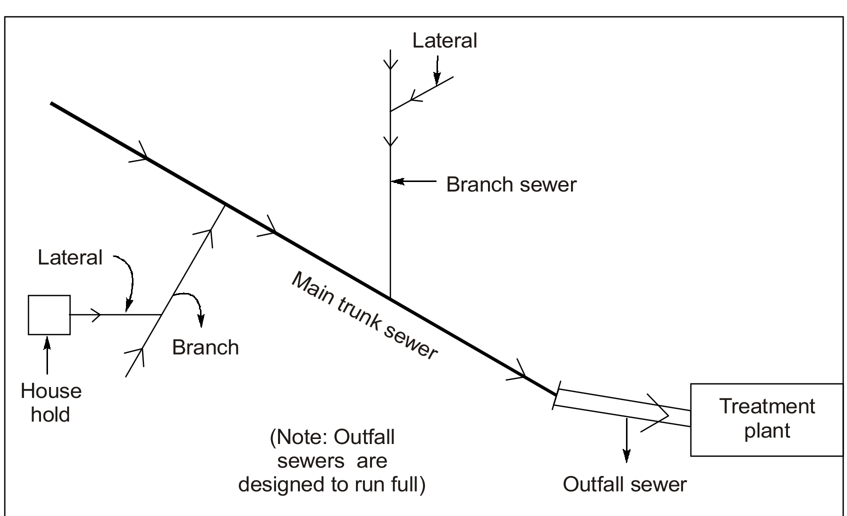


Fig. 6.5 A typical sanitary sewer layout

6.4.2 Storm Sewer

- Storm sewers carry surface runoff developed during or following the period of rainfall over concerned area including street wash.
- Surface water enters a storm drainage system through inlets located in street gutters that collect natural drainage.
- Since no house connection is required, the storm sewers may not depend upon the individual lots, and this may permit them to be run by shorter routes than that of sanitary sewers.
- Storm sewer pipes are set shallower as compared to sanitary sewers as far as possible.

NOTE

Major difference in design of sanitary and storm sewer:

(i) In sanitary and storm sewers the latter are assu-assumed to surcharge and overflow periodically. Sanitary sewer are designed and constructed to prevent surcharging.

(ii) Second difference between sanitary and storm sewers is the pipe size that are needed to serve a given area. Storm drains are larger than the pipe collecting domestic waste water.

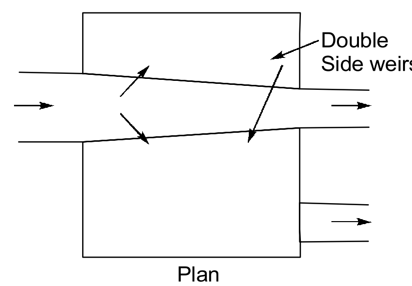


Fig. 6.6 Storm Water Overflow and Sewerage System

6.4.3 Combined Sewer System

- When the drainage is taken along with sewage, it is called Combined Sewer System.
- This system consists of a single sewer line of large diameter through which the sewage and storm water are allowed to flow and are carried to the treatment plant.

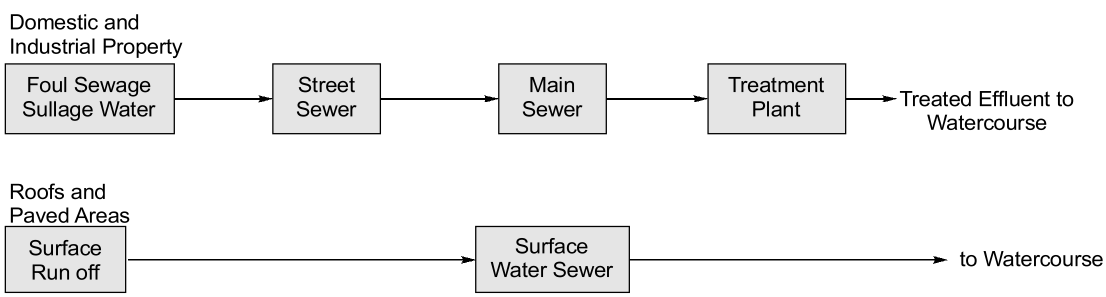


Fig. 6.7 Separate Sewerage System

- The storm water dilutes the sewage and hence its strength is reduced
- In this way, self-cleaning velocity is easily achieved
- As the single sewer line serves the double function, it becomes economical.

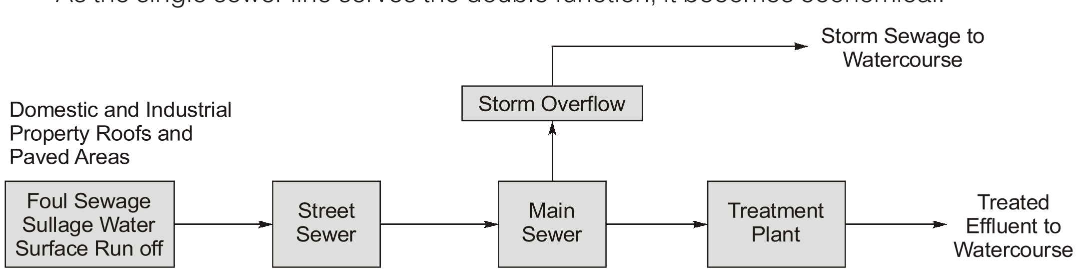


Fig. 6.8 Combined Sewerage System

Types of Sewer in a Typical Collection System

The types and sizes of sewer used will vary with size of the collection system and the location of the waste water- treatment facilities. The principal types of sewers found in most collection system are described by function in table, and illustrated graphically.

Table: 6.1 Types of sewers in a typical collection system

Type of Sewer	Purpose
Building	Building sewers, sometimes called building connections, connect to the building plumbing and are used to convey wastewater from the building to lateral or branch sewers, or any other sewer except another building sewer. Building sewers normally begin outside the building foundation. The distance from the foundation wall to where the sewer begins depends on the local building regulations.
Lateral or branch	Lateral sewers form the first element of a wastewater collection system and are usually in streets or special easements. They are used to collect wastewater from one or more building sewers and convey it to a main sewer.
Main	Main sewers are used to convey wastewater from one or more lateral sewers to trunk sewers or to intercepting sewers.
Trunk	Trunk sewers are large sewers that are used to convey wastewater from main sewers to treatment or other disposal facilities or to large intercepting sewers.
Intercepting	Intercepting sewers are larger sewers that are used to intercept a number of main or trunk sewers and convey the wastewater to treatment or other disposal facilities

DO YOU KNOW? Combined sewer are generally not circular because of generating non-sufficient self cleaning velocity except rainy season. They are egg shaped sewers.

6.5 Assumptions in Sewer Design

Following assumptions are made for the purpose of hydraulic design of Sewer :

- The flow of waste water in sewer is steady and uniform. The unsteady and nonuniform waste water flow characteristics are accounted in the design by proper sizing of man holes.
- The available head in waste water lines is utilized in overcoming surface resistance and, in small part in attaining kinetic energy for the flow.
- The design of sewers are based on peak flow discharge.

Flow Formula:

- Manning's formula is used for open channel flow
- William-Hazen formula is used for closed conduit or pressure flow.

Manning's Formula:

$$V = \frac{1}{n} \times R^{2/3} \times S^{1/2}$$

where, V = velocity in meters/sec

n = Manning's coefficient of roughness

R =Hydraulic radius in meters

S = Slope of hydraulic gradient

William-Hazens Formula:

$$V = 0.849 \times C \times R^{0.63} \times S^{0.54}$$

where, C = William Hazen's coefficient

Table : 6.2 Values of C_H for William Hazen's formula			
S.No	**Type of pipe material**	**Value of C_H for**	
		New pipes	**Design purposes**
1.	Concrete and R.C.C. pipes	140	110
2.	Cast iron pipes	130	100
3.	Galvanised iron popes	120	100
4.	Steel pipes with welded joints	140	100
5.	Steel pipes with rivetted joints	110	95
6.	Steel pipes with welded joints lined with cement for bituminous enamel	140	110
7.	Asbestos cement pipes	150	120
8.	Plastic pipes	150	120

Crimp and Burge's Formula:

$$V = 83.5 \times R^{2/3} \times S^{0.54}$$

This formula is comparable to Manning's formula having $\frac{1}{n} = 83.5$ or $n = 0.012$

Chezy's Formula

$$V = C\sqrt{RS}$$

where, C = Chezy's constant depends upon various factors, such as the size and the shape of the channel, roughness of the channel surface, the hydraulic characteristics of the channel. The value of C can be obtained by using either Kutter's formula or Bazin's formula.

(a) Kutter's Formula

$$C = \frac{\left(23 + \frac{0.00155}{S}\right) + \frac{1}{n}}{1 + \left(23 + \frac{0.00155}{S}\right) \cdot \frac{n}{\sqrt{R}}}$$

where, n = Rugosity coefficient depending upon the type of the channel surface.

Table : 6.3 Mannings or Kutter's Rugosity Coefficients (n)			
S. No.	**Pipe Material**	**Values of *n* at full depth for**	
(1)	**(2)**	**Good interior surface condition (3)**	**Fair interior surface condition (4)**
1.	Salt glazed stoneware pipes	0.012	0.014
2.	Cement concrete pipes	0.013	0.015
3.	Cast iron pipes	0.012	0.013
4.	Brick, unglazed sewers/dains	0.013	0.015
5.	Asbestos cement	0.011	0.012
6.	Plastic (smooth) pipes	0.011	0.011

(b) Bazin's Formula

$$C = \frac{157.6}{1.81 + \frac{k}{\sqrt{R}}}$$ where, k = Bazin's constant

Table : 6.4 Bazin's Constant (K)

S. No. (1)	Type of the inside surface of the sewer or drain (2)	Value of K (3)
1.	Very smooth surfaces.	0.11
2.	Smooth brick and concrete surfaces.	0.29
3.	Rough brick and concrete surfaces.	0.50
4.	Smooth rubble and masonry surfaces.	0.83
5.	Good earthen channels.	1.54
6.	Rough earthen channels.	3.17

6.6 Design Data

- Sanitary sewers are design to run partially full (flow under gravity).
- Sewer should be designed to carry peak discharge i.e. maximum hourly discharge, and should be checked to ensure that at minimum discharge, (i.e. minimum hourly discharge) velocity generated should be greater than self cleansing velocity.
- Self cleansing velocity is the minimum velocity at which no solids get deposited at the bottom of sewer.
- Ratio of maximum discharge to average discharge is maximum in the laterals and decreases progressively from lateral to branch and to main sewer.
- Unless otherwise given, the following data should be adopted for discharges through sewer.

  Maximum hourly discharge = 3 × average daily discharge
  Maximum daily discharge = 2 × average daily discharge
  Maximum hourly discharge = 1/3 × average daily discharge
  Maximum daily discharge = 2/3 × average daily discharge
- It is assumed that almost 80% of water supply reaches sewer. However sewer should be designed to minimum of 100 L/c/d of discharge.
- Self cleansing velocity is the minimum velocity at which not any solids get deposited at the bottom of sewer.
- Self cleansing velocity is given by shield's formula

$$V = \frac{1}{n} R^{1/6} \left[K_s (G_s - 1) d_p\right]^{1/2}$$

where, G_s = Special gravity of particle
d_p = particle size
k_s = a dimension less constant with a value of about 0.04 to start motion of granular particle and about 0.8 for adequate self cleansing of sewer.
R = hydraulic radius of sewer
n = Manning coefficient

Ensuring self cleansing velocity at minimum flow ensures that no solid is deposited even at minimum flow. However, sometimes design is done in such a way that although solid silting may occur at minimum flow, the same should be flushed out at a peak flow.

It is assumed that almost 75%-80% of accounted water supply goes into sewage.

NOTE

- The inorganic sand particles of diameter 1 mm and specific gravity of 2.65 can be removed with a minimum velocity of about 0.45 m/sec.
- Organic particle of 5 mm diameter can also be removed with same minimum velocity.

Maximum Velocity

- The maximum velocity helps in
  - (i) keeping the sewer size under control.
  - (ii) preventing the sewage from getting stale and decomposed by moving it faster, thereby preventing evolution of foul gases.
- To avoid erosion of pipe surface maximum velocity should be limited as follows.

Table : 6.5 Non-scouring Limiting Velocities in Sewers and Drains

S.No.	Sewer Material	Limiting velocity in m/sec
1.	Vitrified tiles and glazed bricks	4.5 – 5.5
2.	Cast iron sewers	3.5 – 4.5
3.	Stone ware sewers	3.0 – 4.0
4.	Cement concrete sewers	2.5 – 3.0
5.	Ordinary brick - lined sewers	1.5 – 2.5
6.	Earthen channels	0.6 – 1.2

- Slope of sewer should be designed for minimum permissible velocity at minimum flow.

6.7 Hydraulic Characteristics of Circular Sections Running Partially Full or full

The circular section is most widely adopted for sewer pipes. They may however, sometimes be of ‘egg shape’ or ‘horse shoe shape’ or ‘rectangular shape’. The circular sewers may sometimes run full or may run partially full. When they run full, their hydraulic properties will be as given below:

Area of cross section

$$A = \frac{\pi}{4}D^2$$

where D is dia of the pipe

wetted perimeter

$$P = \pi D$$

∴ Hydraulic mean depth

$$R = \frac{A}{P} = \frac{\frac{\pi}{4}D^2}{\pi D} = \frac{D}{4}$$

When the sewer run partially full, at a depth, say d, as shown in figure 6.9, the hydraulic elements can be worked out as given below.

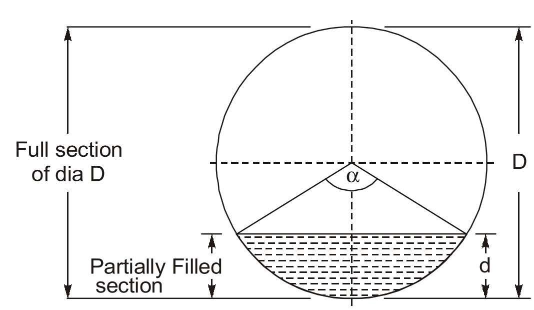


Fig.6.9 Partially filled circular sewer section

The depth at partial flow

$$= d = \frac{D}{2} - \frac{D}{2}\cos\frac{\alpha}{2}$$

where α is the central angle in degrees as shown in figure.

∴ Proportionate depth

$$= \frac{d}{D} = \frac{1}{2}\left(1 - \cos\frac{\alpha}{2}\right) \quad \text{...(i)}$$

Area of cross-section while running partially full

$$= a = \frac{\pi D^2}{4} \times \frac{\alpha}{360^\circ} - \frac{D}{2}\cos\frac{\alpha}{2}\frac{D}{2}\sin\frac{\alpha}{2}$$

$$= \frac{\pi D^2}{4}\left[\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}\right] \quad \text{...(ii)}$$

∴ Proportionate area $= \frac{a}{A} = \left[\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}\right]$ ...(iii)

wetted perimeter, while running partially full

$$= p = \pi D \cdot \frac{\alpha}{360} \quad \text{...(iv)}$$

∴ Proportionate perimeter $= \frac{p}{P} = \frac{\alpha}{360^\circ}$ ...(v)

Hydraulic mean depth (H.M.D), which running partially full

$$= r = \frac{a}{p} = \frac{D}{4}\left[1 - \frac{360^\circ \sin\alpha}{2\pi\alpha}\right] \quad \text{...(vi)}$$

∴ Proportionate H.M.D $= \frac{r}{R} = \left[1 - \frac{360^\circ \sin\alpha}{2\pi\alpha}\right]$ ...(vii)

velocity of flow is given by Manning's formula, as

v = velocity at partial flow

$$= \frac{1}{n} r^{2/3}\sqrt{S_0} \quad [\because S = S_0 \text{ i.e. bed slope}]$$

∴ V = velocity, when running full

$$= \frac{1}{N} \cdot R^{2/3}\sqrt{S_0}$$

[Bed slope $S = S_0$ remains constant whether pipe runs full or partially full]

∴ Proportionale velocity $= \frac{v}{V} = \frac{N}{n} \cdot \frac{r^{2/3}}{R^{2/3}}$ ...(viii)

Assuming that roughness coefficient n does not vary with depth,

we have $n = N$

∴ Proportionate velocity $= \frac{v}{V} = \frac{r^{2/3}}{R^{2/3}} = \left[1 - \frac{360^\circ \sin\alpha}{2\pi\alpha}\right]^{\frac{2}{3}}$ ...(ix)

Since, discharge is given by a.v, therefore

Discharge when pipe is running partially full

$$= q = av \quad \text{...(x)}$$

Discharge when pipe is running full

$$= Q = AV \quad \text{...(xi)}$$

∴ Proportionate discharge $= \boxed{\frac{q}{Q} = \frac{av}{AV} = \frac{a}{A} \cdot \frac{v}{V}} = \boxed{\frac{q}{Q} = \left[\frac{\alpha}{360^\circ} - \frac{\sin\alpha}{2\pi}\right]\left[1 - \frac{360^\circ \sin\alpha}{2\pi\alpha}\right]^{2/3}}$ ...(xii)

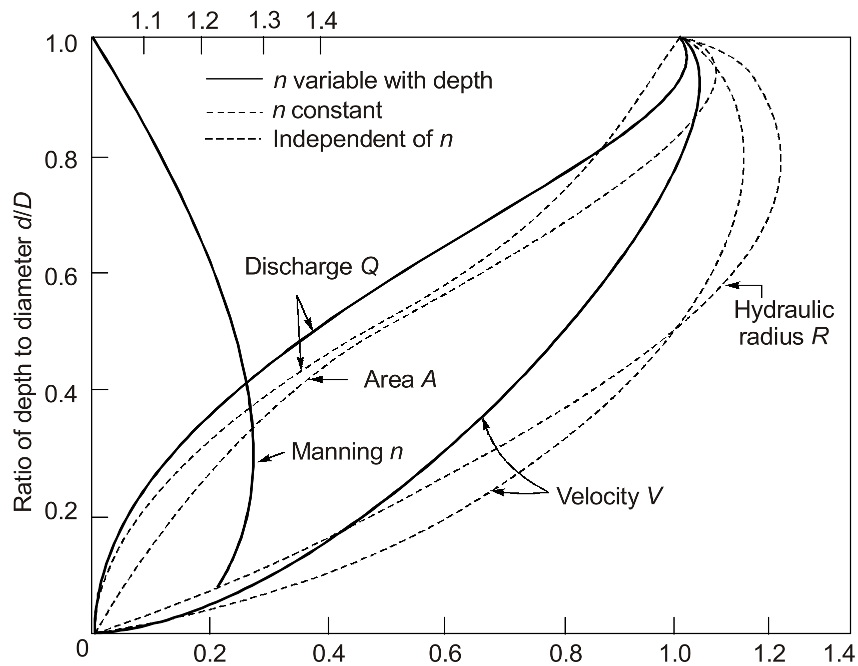


Fig. 6.10 Hydraulic elements for circular sewers

By knowing the conditions under which sewer runs full and by knowing two ratios by hydraulic elements under partial and full flow conditions, third can be calculated analytically or by using partial flow diagram.

Remember

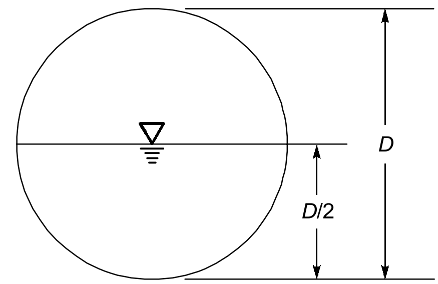
- If Manning's coefficient 'n' is assumed constant with depth then $\frac{v}{V} = \left(\frac{r}{R}\right)^{2/3}$
- For constant 'n', velocity of flow is maximum when $\frac{d}{D} = 0.81$ and this V_{max} is 12.5% greater than when running full.
- For constant 'n', discharge is maximum when $\frac{d}{D} = 0.95$ and this q_{max} is 7% greater than the discharge at running full.
- $\frac{v}{V}$ decreases less sharply than $\frac{q}{Q}$ below $\frac{1}{2}$ full depth (for constant n)
- As $\frac{d}{D} < 0.5$, the decline in velocities is not so sharp as the decline in discharges, because the area (on which discharge depends) reduces much faster as compared to the hydraulic mean depth (on which velocity depends).
- $\frac{q}{Q} = \frac{1}{2}$ at $\frac{1}{2}$ full flow (n = constant)
- If $\frac{q}{Q} \to 0.5$ then $\frac{v}{V} \geq 1$ (for n = constant)
- For most efficient system, design should be done for 3 times of average and self cleaning velocity should be checked for $\frac{1}{3}$rd of daily discharge.

Example - 6.1 Determine the size of a circular sewer for a discharge of 500 litres per second running half full, Assume = S = 0.0001

Solution:

$$d = 0.5\,D$$

When sewer running half full



$$a = \frac{\pi}{8}D^2$$

$$p = \frac{\pi D}{2}$$

$$r = \frac{a}{p} = \frac{\frac{\pi}{8}D^2}{\frac{\pi D}{2}} = \frac{D}{4}$$

Given, $Q = 0.5\ \text{m}^3/\text{sec}$

by Manning's furmula

$$Q = \frac{1}{n}r^{2/3}.s^{1/2}\ \text{(a)}$$

$$0.5 = \frac{1}{0.015}\times\left(\frac{D}{4}\right)^{2/3}\times(0.0001)^{1/2}\times\frac{\pi}{8}D^2$$

$$D^{8/3} = 4.8128$$

$$D = (4.8128)^{3/8} \simeq 1.80\ \text{m}$$

6.7.1 Equal Degree fo Self Cleansing

If in a particular sewer, drag force under partial flow and drag force under full flow is to be same then,

$$\gamma.RS = \gamma.rS_0$$

$$RS = rS_0$$

Under this condition

$$\frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{2/3}\left(\frac{S_0}{S}\right)^{1/2}$$

$$\frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{2/3}\left(\frac{1}{r/R}\right)^{1/2}$$

$$\frac{v}{V} = \frac{N}{n}\left(\frac{r}{R}\right)^{1/6}$$

NOTE

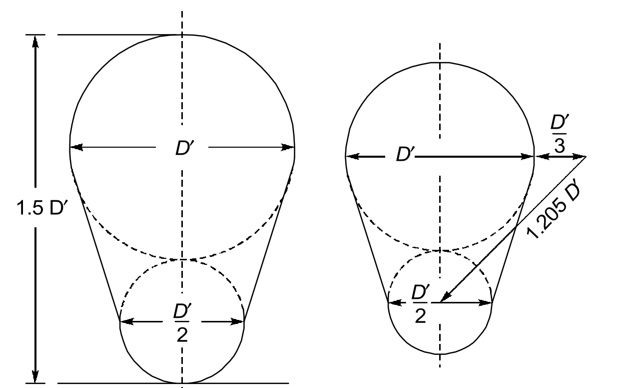
When minimum velocity requirement in a sewer is not satisfied option is to

(a) increase the slope.

(b) increase the diameter of sewer.

6.8 Egg Shaped Sewer

- Circular sewer sections are mostly used for separate sewage system.
- The circular sections are generally preferred to all other shapes because of their following advantages:
  - (i) They can be manufactured most easily and conveniently.
  - (ii) A circular sewer provides the maximum area for a given perimeter and thus providing the maximum hydraulic mean depth when running full or half full, and is therefore, the most efficient section of these flow conditions.
  - (iii) Circular section utilizes the minimum quantities of materials and is therefore the cheapest and most economical.
  - (iv) A circular section, being of uniform curvature all round, offers less opportunities for deposits.
- All these advantages of circular sections are obtained only when the sections runs at least half full. When the depth goes below half depth the velocity reduces considerably.
- If a circular sewer is used for combined system it will be effective only during maximum rain water flow but during dry weather flow, velocity generated would be very less. Thus to take advantage of a circular sewer, two such circular sewer are assumed to be combined into one to form an "egg shaped sewer" in which smaller circular portion will be effective during dry weather and full section is effective during maximum rain water flow.



(a) "Standard" or "Metropolitan" Egg shaped section. (b) "New" Egg shaped section.

Fig. 6.11 Typical view of Egg Shaped Section

- Two sewers of different shapes are said to be hydraulically equivalent when they discharge at the same rate, while flowing full, on the same grade.
- The egg shaped sewer of an equivalent section, whose top diameter $D' = 0.84\,D$, when D = diameter of circular sewer of same cross-section are obtained for passing the requisite discharge.
- Their disadvantage over circular sewer are:
  - (i) They are more difficult to construct.
  - (ii) Since the smaller base has to support the weight of the upper boarder section, they are less stable.
  - (iii) They require more material and are, therefore, more costly.

NOTE In combined sewers, the variation in discharge could be as large as 20 to 25 times. Egg shaped sewer produces 2 to 15% higher velocity than that provided by hydraulically equivalent circular sewer.

6.9 Storm Water Drainage

- Design of storm water drainage requires the estimation of peak runoff rate for design.
- Peak runoff depends on the intensity of rainfall.
- The intensity of rainfall depends on recurrence interval and duration of rainfall.

NOTE Rain of larger recurrence interval will have higher intensity. Hence 5 year recurrence interval rain is chosen for design, then it may lead to flooding in every 5-year (because the rainfall has a probability of exceeding the adopted value every 5-years and drainage has not been designed for this large rainfall). If larger frequency of rainfall is adopted in design, larger sewer will be required to carry the runoff safely.

6.9.1 Computing the peak drainage discharge by the use of Rational Formula

6.9.1.1 Time of Concentration

- If a rainfall is applied to an impervious surface at a constant rate, the resultant runoff from the surface would finally reach a rate equal to the rainfall. In the beginning, only a certain amount of water will reach the outlet, but after sometime the water will start reaching the outlet from the entire area, and in this case, the runoff rate would become equal to the rate of rainfall. "The period after which the entire area will start contributing to the runoff is called the Time of concentration"
- The time of concentration for a given storm water drain generally consist of two parts.
    1. The inlet time or overland flow time or time of equilibrium (T_i). The time taken by the water to flow overland from the critical point upto the point when it enters the drain mouth.

$$T_i = \left(0.885\frac{L^3}{H}\right)^{0.385}$$

where, T_i = Inlet time in hours
L = length of overland flow in km from the critical point to the mouth of the drain.
H = Total fall of level from the critical point to the mouth of the drain in meters.

    2. The channel flow time or gutter flow time (T_f) i.e. the time taken by the water to flow in the drain channel from the mouth to the considered point. This may be obtained by dividing the length of the drain by the flow velocity in the drain.

$$T_f = \frac{\text{Length of the drain}}{\text{Velocity in the drain}}$$

The total time of concentration (T_c) = $T_i + T_f$

6.9.1.2 Rain Fall Intensity

The intensity of rainfall during this much of time (for the given design frequency) can be easily obtained from the standard intensity duration curves or DAD curves.

The value of intensity so obtained is still the rainfall intensity at the rain gauge station and is called the "point rainfall intensity". In order to make it effective over the entire catchment area, it is necessary to multiply it by a factor called dispersion factor or areal distribution factor. (Fig. 6.12)

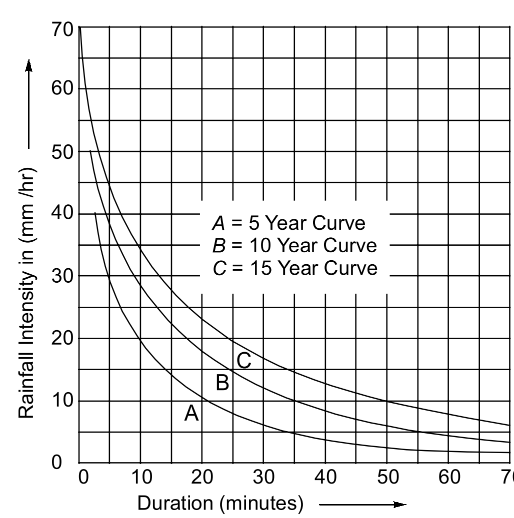


Fig. 6.12 Typical Intensity Duration Curves

Thus design rainfall intensity = (point rainfall intensity) × (area dispersion factor)

- In absence of standard intensity duration curves, the value of design rainfall intensity (p_c)

$$p_c = p_0\left(\frac{2}{1+T_c}\right)$$

p_0 = (point rainfall intensity of a particular frequency) × (area dispersion factors)
This point rainfall intensity is obtained from contour map or maximum rainfall of a particular frequency.
p_0 is in cm/hr
p_c is in cm/hr
T_c in hr = Time of concentration

- The design rainfall can also be obtained from the formula.

$$p = \frac{a}{T+b}$$

where, p = Rain intensity in cm/hr
T = Time in minutes
a and b = Constants

$$p = \frac{75}{T_c+10} \quad \text{(for } T \text{ varying between 5 to 20 minute)}$$

and

$$p = \frac{100}{T_c+20} \quad \text{(for } T \text{ varying between 20 to 100 minutes)}$$

6.9.2 Rational Formula

$$Q = C \cdot p \cdot A$$

where, Q = Peak rate of runoff (m^3/s)
p = Runoff coefficient $\frac{\text{runoff}}{\text{rain flow}}$
i = intensity of rainfall (m/s)
A = drainage basin area-catchment area (m^2)

The coefficient of Runoff (C) is in fact, the impervious factor of runoff, representing the ratio of precipitation to runoff. Runoff coefficient value increase as imperiousness of catchment increases. When several different types of surfaces comprise the catchment than weighted average value of runoff coefficient is adopted.

$$C_{av} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + ...}{A_1 + A_2 + A_3 + ...}$$

A_1 area has runoff coefficient = C_1
A_2 area has runoff coefficient = C_2
A_3 area has runoff coefficient = C_3 etc.

NOTE

1. Rainfall intensity depends on the storm recurrence interval and the storm duration.
2. In intensity duration curves, if 15 hr recurrence interval rain is to be adopted and time of concentration is 25 minutes then rain fall intensity to be adopted will be 25 mm/hr.

Example - 6.2 The surface water from airport road side is drained to the longitudinal side drain from across one half of a bituminous pavement surface of total width 7.0 m, shoulder and adjoining land of width 8.0 m on one side of the drain. On other side of the drain, water flows across from reserve land with average turf and 2% cross slope towards the side drain, the width of the strip of land being 25 m. The inlet time may be assumed to be 10 min for these conditions. The runoff coefficients of the pavement, shoulder and reserve land with turf are 0.8, 0.25 and 0.35 respectively. The length of the stretch of land parallel to the road from where the water is expected to flow to the side drain is 400 m. Estimate the quantity of runoff flowing in the drain assuming 10 year frequency. The side drain will pass through clayey soil with allowable velocity of flow as 1.33 m/s. Intensity duration chart for 10 year frequency is

Duration (minutes)	5	10	15	20	30
Intensity (mm/hr)	160	150	125	110	95

Solution: Average runoff factor K for the entire area contributing discharge

$$K = \frac{K_1A_1 + K_2A_2 + K_3A_3}{A_1 + A_2 + A_3} = \frac{0.8\times(7\times400)+0.25\times(8\times400)+0.35\times(25\times400)}{(7\times400)+(8\times400)+(25\times400)}$$

$$= \frac{6540}{2800+3200+10000} = 0.40875$$

Total area $= 16000\ m^2 = 1.6$ ha

Time of concentration

$$T_C = T_i + T_f$$

$$T_f = \text{channel flow time in drain} = \frac{400}{1.33} = 300 \text{ sec} = 5 \text{ min}$$

$$\therefore \quad T_C = 10 + 5 = 15 \text{ min}$$

Corresponding of T_C, rainfall intensity = 125 mm/hr

$$\Rightarrow \quad P_C = 12.5 \text{ cm/hr}$$

$$\text{Peak discharge } Q_P = \frac{1}{36} K P_C A = \frac{1}{36}\times 0.40875\times12.5\times1.6 = \mathbf{0.227\ m^3/s}$$

Example - 6.3 Find the relation between the side of a square section of one sewer and the diameter of a circualr section of another sewer when both are hydraulically equipment.

Solution:

Let D be the diameter of the circular sewer and be the side of the equivalent square sewer. The discharging capacity of circular sewer while running full at a gradient of 1 in S.

$$Q_C = \frac{1}{N}\left(\frac{\pi}{4}D^2\right)\left(\frac{D}{4}\right)^{2/3}(S)^{1/2} \quad \text{...(i)}$$

The discharging capacity of the square section while running full, at a gradient of 1 in S

$$Q_S = \frac{1}{N}(b^2)\left(\frac{b^2}{4b}\right)^{2/3}(S)^{1/2} \quad \text{...(ii)}$$

For hydraulically equivalent section

$$Q_C = Q_S$$

$$\therefore \quad \frac{1}{N}\left(\frac{\pi}{4}D^2\right)\left(\frac{D}{4}\right)^{2/3}\sqrt{S} = \frac{1}{N}b^2\left(\frac{b}{4}\right)^{2/3}\sqrt{S}$$

$$\frac{\pi}{4}\frac{D^{8/3}}{2.52} = \frac{b^{8/3}}{2.52}$$

$$D^{8/3} = 1.272\, b^{8/3}$$

$$D = (1.272)^{8/3}\, b$$

$$D = 1.094\, b$$