## CHAPTER 3: DESIGN AND CONSTRUCTION OF SEWERS

### 3.1 GENERAL

The major roles of a sewer system can be listed as follows:

– Improvement in the environment by removing the sewage as it originates

– Preventing inundation of low lying areas that may be otherwise caused by not providing sewers

– Prevention of vector propagation by sewage stagnations

– Avoiding cross connections with freshwater sources by seepage

In addition, there is a strong emphasis on:

a) Avoiding sewer impacts on groundwater quality by infiltration of soil water into sewers and exfiltration of sewage into soil water, occurring rather as a cycle depending on the flow conditions in leaky sewers, and

b) Moving away from the mind-set that a sewer system shall necessarily be an underground sewer right in the middle of the road with costly construction, upkeep and remediation and making the objective realizable if necessary in an incremental sewerage commensurate with optimizing the area coverage in the available financial and human resources to create and sustain the system.

This chapter presents the following:

Part - 1   Estimation of Design Flows

Part - 2   Types and Hydraulics of Sewers

Part - 3   Design of Sewer Networks

Part - 4   Types and Construction of Manholes

Part - 5   Laying, Jointing and Construction of Sewers

### PART - 1  ESTIMATION OF DESIGN FLOWS

### 3.2 DESIGN PERIOD

The length of time up to which the capacity of a sewer will be adequate is referred to as the design period. In fixing a design period, consideration must be given for the useful life of structures and equipment employed, taking into account obsolescence as well as wear and tear. The flow is largely a function of the population served, population density, water consumption, lateral and sub main sewers are usually designed for peak flows of the population at saturation density as set forth in the master plan. Trunk sewers, interceptors, and outfalls are difficult and uneconomical to be enlarged or duplicated and hence are designed for longer design periods. In the case of trunk sewers serving relatively undeveloped areas adjacent to metropolitan areas, it is advisable to construct initial facilities for more than a limited period. Nevertheless, right of way for future larger trunk sewers can be acquired or reserved. The recommended design period for various components shall be as in Table 2-1.

## 3.3 POPULATION FORECAST

Methods of estimation of population for arriving at the design population have been discussed in Section 2.6. When a master plan containing land use pattern and zoning regulations is available for the town, the anticipated population can be based on the ultimate densities and permitted floor space index provided for in the master plan.

In the absence of such information on population, the following densities are suggested for adoption as in Table 3.1.

Table 3.1 Densities of Population vs. Populated areas

| Size of town (Population) | Density of population per hectare |
|---|---|
| Up to 5,000 | 75-150 |
| Above 5,001 to 20,000 | 150-250 |
| Above 20,001 to 50,000 | 250-300 |
| Above 50,001 to 1,00,000 | 300-350 |
| Above 1,00,001 | 350-1,000 |

Source: CPHEEO, 1993

In cities where Floor Space Index (FSI) or Floor Area Ratio (FAR) limits are fixed by the local authority this approach may be used for working out the population density. The FSI or FAR is the ratio of total floor area (of all the floors) to the plot area.

The densities of population on this concept may be worked out as in the following example for an area of one hectare (ha)

| | |
|---|---|
| Roads | 20% |
| Gardens | 15% |
| Schools (including playgrounds) | 5% |
| Markets | 2% |
| Hospital and Dispensary | 2% |
| Total | 44% |

Area available for Residential Development = 100 - 44 = 56% or 0.56

Actual total floor area = Area for residential development x FSI

Assuming an FSI of 0.5 and floor area of 9 $m^2$/person

Number of persons or density per hectare $= \dfrac{0.56 \times 10,000 \times 0.5}{9} = 311$

## 3.4    TRIBUTARY AREA

The natural topography, layout of buildings, political boundaries, economic factors etc., determine the tributary area. For larger drainage areas, though it is desirable that the sewer capacities be designed for the total tributary area, sometimes, political boundaries and legal restrictions prevent the sewers to be constructed beyond the limits of the local authority. However, in designing sewers for larger areas, there is usually an economic advantage in providing adequate capacity initially for a certain period of time and adding additional sewers, when the pattern of growth becomes established. The need to finance projects within the available resources necessitates the design to be restricted to political boundaries. The tributary area for any section under consideration has to be marked on a key plan and the area can be measured from the map.

## 3.5    PER CAPITA SEWAGE FLOW

The entire spent water of a community should normally contribute to the total flow in a sanitary sewer. However, the observed dry weather flow quantities usually are slightly less than the per capita water consumption, since some water is lost in evaporation, seepage into ground, leakage etc. In arid regions, mean sewage flows may be as little as 40% of water consumption and in well developed areas; flows may be as high as 90%. However, the conventional sewers shall be designed for a minimum sewage flow of 100 litres per capita per day or higher as the case may be. Non-conventional sewers shall be designed as the case may be.

For some areas, it is safe to assume that the future density of population for design as equal to the saturation density. It is desirable that sewers serving a small area be designed accordingly on saturation density.

For new communities, design flows can be calculated based on the design population and projected water consumption for domestic use, commercial use and industrial activity. In case a master plan containing land use pattern and zoning regulation is available, the anticipated population can be based on the ultimate densities as in Table 3.1.

The flow in sewers varies from hour to hour and seasonally. However, for the purpose of hydraulic design estimated peak flows are adopted. The peak factor or the ratio of maximum to average flows depends upon contributory population as given in Table 3.2.

Table 3.2 Peak factor for Contributory Population

| Contributory Population | Peak Factor |
|---|---|
| up to 20,000 | 3.00 |
| Above 20,001 to 50,000 | 2.50 |
| Above 50,001 to 7,50,000 | 2.25 |
| above 7,50,001 | 2.00 |

Source: CPHEEO, 1993

The peak factor also depends upon the density of population, topography of the site, hours of water supply and hence, individual cases may be further analyzed if required. The minimum flow may vary from 1/3 to 1/2 of average flow.

## 3.6    INFILTRATION

Estimate of flow in sanitary sewers may include certain flows due to infiltration of groundwater through joints. Since sewers are designed for peak discharges, allowances for groundwater infiltration for the worst condition in the area should be made as in Table 3.3

Table 3.3  Ground water infiltration

|  | Minimum | Maximum |
|---|---|---|
| Litres/ha/day | 5,000 | 50,000 |
| Litres/km/day | 500 | 5,000 |
| Litres/day/manhole | 250 | 500 |

Source: CPHEEO, 1993

Once the flow is estimated as per Table 3.3, the design infiltration value shall be limited to a maximum of 10% of the design value of sewage flow.

Care shall be taken that in high ground water locations and coastal locations, the sewer pipes shall not be stoneware or vitrified clay pipes and instead shall be cast iron / ductile iron pipes or other non-metallic pipes with safeguards against floatation as discussed later in the section on laying of sewers.

## 3.7    SEWAGE FROM COMMERCIAL INSTITUTIONS

The industries and commercial buildings often use water other than the municipal supply and may discharge their liquid wastes into the sanitary sewers. Estimates of such flows have to be made separately as in Table 3.4 (overleaf) for their potable water supply.

## 3.8    INDUSTRIAL EFFLUENTS TO BE DISCOURAGED

The mixing of industrial effluents through discharge into public sewers is undesirable due to the possible detrimental effects of such effluent on the operation of biological sewage treatment process. This aspect has been well recognized in recent times and industrial areas having polluting industries are generally located such as to avoid mixing with sewage.

However, in cities that have undergone unregulated growth in the past, polluting industries may exist in pockets of mixed land use. In such cases, those industries are required to implement zero liquid discharge (ZLD) by reusing the effluents after appropriate treatment in house.

Of all the industries, this shall strictly apply to the automobile service stations and machine shops from where the spent metal plating baths and oil & grease shall be prevented from entering the sewers.

Table 3.4  Institutional needs for potable water

| No. | Institutions | Water Supply (litres) |
|---|---|---|
| 1 | Hospital including laundry and beds exceeding 100 | 450 per bed |
| 2 | Hospital including laundry and beds not exceeding 100 | 340 per bed |
| 3 | Lodging houses / hotels | 180 per bed |
| 4 | Hostels | 135 lpcd |
| 5 | Nurses homes and medical quarters | 135 lpcd |
| 6 | Boarding schools/colleges | 135 lpcd |
| 7 | Restaurants | 70 per seat |
| 8 | Airports and Seaports, duty staff | 70 lpcd |
| 9 | Airports and Seaports, alighting and boarding persons | 15 lpcd |
| 10 | Train and Bus stations, duty staff | 70 lpcd |
| 11 | Train and Bus stations, alighting and boarding persons | 15 lpcd |
| 12 | Day schools/colleges | 45 lpcd |
| 13 | Offices | 45 lpcd |
| 14 | Factories, duty staff | 45 lpcd |
| 15 | Cinema, concert halls and theatres | 15 lpcd |

## 3.9    STORM RUNOFF

The sanitary sewers are not expected to receive storm water. Strict inspection, vigilance, and proper design and construction of sewers and manholes should eliminate this flow or bring it down to a very insignificant quantity.

However, in small habitations where rainfall is almost a continuous affair, it may be necessary to include storm water in the design of sewers as under.

### 3.9.1   Estimation of Storm Runoff

The storm runoff is that portion of the precipitation, which drains over the ground. Estimation of such runoff reaching the storm sewers therefore is dependent on the intensity, duration of precipitation, characteristics of the tributary area, and the time required for such flow to reach the sewer.

The design of storm water sewers begins with an estimate of the rate and volume of surface runoff. When rain falls on a given catchment, a portion of the precipitation is intercepted by the vegetation cover that mostly evaporates, a portion hits the soil and some of it percolates down below and the rest flows over the ground. The higher the intensity of rain, the higher will be the peak runoff.

The characteristics of the drainage area such as imperviousness, topography including depressions, water pockets, shape of the drainage basin and duration of the precipitation determine the fraction of the total precipitation, which will reach the sewer. This fraction is known as the coefficient of runoff.

The time-period after which the entire area begins contributing to the total runoff, at a given monitoring point, is known as the time of concentration. It is also defined as the time it takes for a drop of water to flow from the most distant point to the outlet of the basin. The duration of rainfall that is equal to the time of concentration is known as the critical rainfall duration. The rational formula for the relationship between peak runoff and the rainfall is given below.

$$Q = 10\,C\,i\,A \tag{3.1}$$

where,

Q : Runoff in $m^3$/hr

C : Dimensionless runoff coefficient

i : Intensity of rainfall in mm/hr

A : Area of drainage district in hectares

The storm water flow for this purpose may be determined by using the rational method, hydrograph method, rainfall-runoff correlation studies, digital computer models, inlet method or empirical formulae. The empirical formulae that are available for estimating the storm water runoff can be used only when comparable conditions to those for which the equations were derived initially exist.

A rational approach, therefore, demands a study of the existing precipitation data of the area concerned to permit a suitable forecast. Storm sewers are not designed for the peak flow of rare occurrence such as once in 10 years or more, but it is necessary to provide sufficient capacity to avoid too frequent flooding of the drainage area. There may be some flooding when the precipitation exceeds the design value, which has to be permitted. The frequency of such permissible flooding may vary from place to place, depending on the importance of the area. Though such flooding causes inconvenience, it may have to be accepted occasionally, considering the economy effected in the sizes of the drains and the costs.

The maximum runoff, which has to be carried in a sewer section should be computed for a condition when the entire basin draining at that point becomes contributory to the flow and the time needed for this is known as the time of concentration (with reference to the concerned section). Thus, for estimating the flow to be carried in the storm sewer, the intensity of rainfall which lasts for the period of time of concentration is the one to be considered contributing to the flow of storm water in the sewer. Of the different methods, the rational method is more commonly used as herein.

It may be reiterated that Q represents only the maximum discharge caused by a particular storm. The portion of rainfall, which finds its way to the sewer, is dependent on the imperviousness and the shape of the drainage area apart from the duration of storm.

The percent imperviousness of the drainage area can be obtained from the records of a particular district. In the absence of such data, Table 3.5 (overleaf) may serve as a guide.

Table 3.5  Percentage of Imperviousness of Areas

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

Source: CPHEEO, 1993

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:

$$I = [(A_1 I_1) + (A_2 I_2) + \ldots + (A_n I_n)] / [(A_1 + A_2 + \ldots + A_n)] \qquad (3.2)$$

where,

$I$ :        Weighted average imperviousness of the total drainage basin

$A_1, A_2, A_n$ :      Sub drainage areas

$I_1, I_2, I_n$ :       Imperviousness of the respective sub-areas.

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 3.6 (overleaf).

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.

### 3.9.2    Rational Method

### 3.9.2.1    Runoff-Rainfall Intensity Relationship

The entire precipitation over the drainage district does not reach the sewer. The characteristics of the drainage district, such as, imperviousness, topography including depressions and water pockets, shape of the drainage basin and duration of the precipitation determine the fraction of the total precipitation, which will reach the sewer.

This fraction known as the coefficient of runoff needs to be determined for each drainage district. The runoff reaching the sewer is given by Equation (3.1).

### 3.9.2.2    Storm Frequency

The frequency of storm for which the sewers are to be designed depends on the importance of the area to be drained. Commercial and industrial areas have to be subjected to less frequent flooding. The suggested frequency of flooding in the different areas is as follows -:

Table 3.6  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 | 0.700 | 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: CPHEEO, 1993

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

### 3.9.2.3   Intensity of Precipitation

The intensity of rainfall decreases with duration. Analysis of the observed data on intensity and duration of rainfall of past records over a period of years in the area is necessary to arrive at a fair estimate of intensity-duration for given frequencies. The longer the record available, the more dependable is the forecast. In Indian conditions, intensity of rainfall adopted in design is usually in the range of 12 mm/hr to 20 mm/hr or based on the actual record.

Table 3.7 gives the analysis of the frequency of storms of stated intensities and durations during 26 years for which rainfall data were available for a given town.

Table 3-7  Duration vs. intensity of storms

| Duration in Minutes | Intensity mm/hr | 30 | 35 | 40 | 45 | 50 | 60 | 75 | 100 | 125 |
|---|---|---|---|---|---|---|---|---|---|---|
| | | No. of storms of stated intensity or more for a period of 26 years | | | | | | | | |
| 5 | | | | | | 100 | 40 | 18 | 10 | 2 |
| 10 | | | | 90 | 72 | 41 | 25 | 10 | 5 | 1 |
| 15 | | | 82 | 75 | 45 | 20 | 12 | 5 | 1 | |
| 20 | | 83 | 62 | 51 | 31 | 10 | 9 | 4 | 2 | |
| 30 | | 73 | 40 | 22 | 10 | 8 | 4 | 2 | | |
| 40 | | 34 | 16 | 8 | 4 | 2 | 1 | | | |
| 50 | | 14 | 8 | 4 | 3 | 1 | | | | |
| 60 | | 8 | 4 | 2 | 1 | | | | | |
| 90 | | 4 | 2 | | | | | | | |

Source: CPHEEO, 1993

The stepped line indicates the location of the storm occurring once in 2 years, i.e., 13 times in 26 years. The time-intensity values for this frequency are obtained by interpolation from Table 3.8.

Table 3.8 Time intensity values of storms

| i (mm/hr) | t (min) | i (mm/hr) | t (min) |
|---|---|---|---|
| 30 | 51.67 | 50 | 18.50 |
| 35 | 43.75 | 60 | 14.62 |
| 40 | 36.48 | 75 | 8.12 |
| 45 | 28.57 | | |

Source: CPHEEO, 1993

The relationship may be expressed by a suitable mathematical formula, several forms of which are available. The following two equations are commonly used:

i) $\quad i = \dfrac{a}{\left(t^{n}\right)}$ $\qquad\qquad$ (3.3)

ii) $\quad i = \dfrac{a}{t+b}$ $\qquad\qquad$ (3.4)

where,

$\quad i:$ $\qquad\qquad$ Intensity of rainfall (mm/hr)

$\quad t:$ $\qquad\qquad$ Duration of storm (minutes)

$\quad a, b$ and $n:$ $\quad$ Constants

The available data on i and t are plotted and the values of the intensity (i) can then be determined for any given time of concentration, (tc).

### 3.9.2.4    Time of Concentration

It is the time required for the rain-water to flow over the ground surface from the extreme point of the drainage basin and reach the point under consideration. It is equal to inlet time (t) plus the time of flow in the sewer (tf).

The inlet time is dependent on the distance of the farthest point in the drainage basin to the inlet manhole, the shape, characteristics and topography of the basin and may generally vary from 5 to 30 minutes. In highly developed sections, the inlet time may be as low as 3 minutes. The time of flow is determined by the length of the sewer and the velocity of flow in the sewer. It is to be computed for each length of sewer to be designed.

a)  Tributary Area

For each length of storm sewer, the drainage area should be indicated clearly on the map and measured. The boundaries of each tributary are dependent on topography, land use, nature of development and shape of the drainage basins. The incremental area may be indicated separately on the compilation sheet and the total area computed.

b) Duration of Storm

Continuously long, light rain saturates the soil and produces higher coefficient than that due to, heavy but intermittent, rain in the same area because of the lesser saturation in the latter case. The runoff from an area is significantly influenced by the saturation of the surface to nearest the point of concentration, rather than the flow from the distant area. The runoff coefficient of a larger area has to be adjusted by dividing the area into zones of concentration and by suitably decreasing the coefficient with the distance of the zones.

A typical example of the computation of storm runoff is given in Appendix A.3.1.

## 3.10    MEASUREMENT OF FLOWS IN EXISTING DRAINS/SEWERS

Quite often, the measurement of flows in existing drains or sewers will provide valuable data for a more realistic assessment of the design flows. In general, non-sewered areas will most certainly be having a set of drains where the generated sewage will be flowing out. The assessment of the flows in drains can be done by a variety of methods right from the rudimentary crude method to the most sophisticated dye tracer method. The choice of methods presented hereunder is considered to be appropriate to the conditions in the country particularly, away from metropolitan centres.

a) The Float Method

At the very outset, a non-intrusive method is called for. This can be done by finding out the time taken for a float like an empty match-box or a plastic box to travel for about 3 m in a straight reach and measuring the width and depth of flow in the drain. If we assume the respective values as 20 seconds, the width as 0.9 m and depth of flow as 0.6 m, the flow can be assessed as 0.8 × 0.9 × 0.6 × (3/20) × 1000 = 65 lps. The factor of 0.8 is the average velocity in such drains for the depth of flow.

b) The V notch method

This requires the insertion of a V notch plate in the drain at a location where the downstream discharge can be a free fall. These plates can be cut out from stainless steel (SS) or Teflon sheets of nominal thickness of about 2 mm and inserted tightly into the drain and the gaps can be closed by a mixture of clay and cement in equal proportion mixed to a thick consistency and smeared on the downstream side. The V notch is best chosen such that the angle subtended is 90 degrees. The clearances to be ensured are shown in Figure 3.1
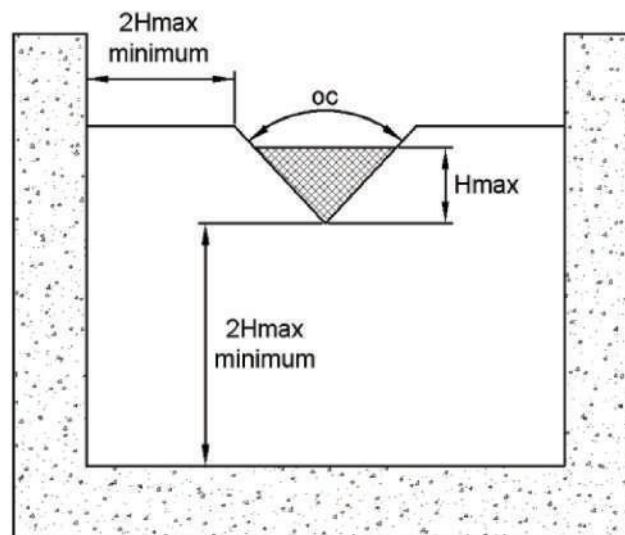


Figure 3.1  Typical mounting of a V Notch in a drain

The depth of flow is measured over the lower tip of the V bottom and the discharge is

$$Q = 1.42 \times \tan \text{ of angle of V notch} \times H^{2.5} \tag{3.5}$$

As the angle is 90 degrees, the tangent is equal to 1 and hence, the equation simplifies to

$$Q = 1.42 \times H^{2.5} \tag{3.6}$$

Where Q is cum/sec and H is in m

c) The rectangular weir method

This can be used if there is already an existing levelled overflow weir like the overflow culverts in irrigation canals. In smaller drains and in places where workmanship of V notch cuts are difficult, these can be used easily by cutting a mild steel or wood sheet as shown in Figure 3.2.
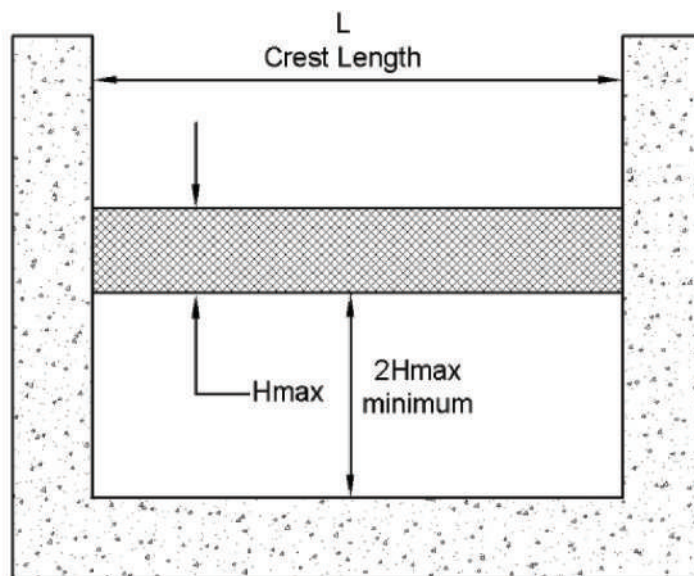


Figure 3.2  Typical mounting of a rectangular weir in a drain

The depth of flow is measured over the overflow edge of the notch and the discharge is

$$Q = 1.85 \times L \times H^{1.5} \tag{3.7}$$

Where,
　　Q is cum/sec,
　　H is in m,
　　L is the length of weir

d) The rectangular weir with end contractions method

These are similar to the rectangular weir except that the length of the weir is smaller than the width of the drain as in Figure 3-3 overleaf.

The depth of flow is measured over the overflow edge of the notch and the discharge is

$$Q = 1.85 \times (L - 0.2H) \times H^{1.5} \tag{3.8}$$

Where
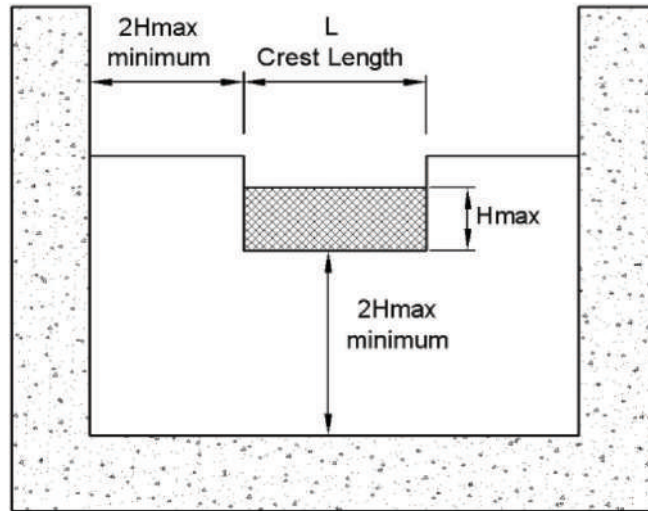　　Q is cum/sec,
　　H is in m,
　　L is the length of weir

Figure 3.3  Typical mounting of a rectangular weir with end constrictions in a drain

e) The Palmer-Bowlus Flume

This can be used in case of both the drains and pipes flowing under gravity. Its major advantages are (i) less energy loss; (ii) minimal restriction to flow and (iii) Easy installation in existing conduits. It is a readymade piece for various widths and diameters. The placement in a drain will be as in Figure 3.4 and that in a sewer pipe will be as in Figure 3.5 overleaf.
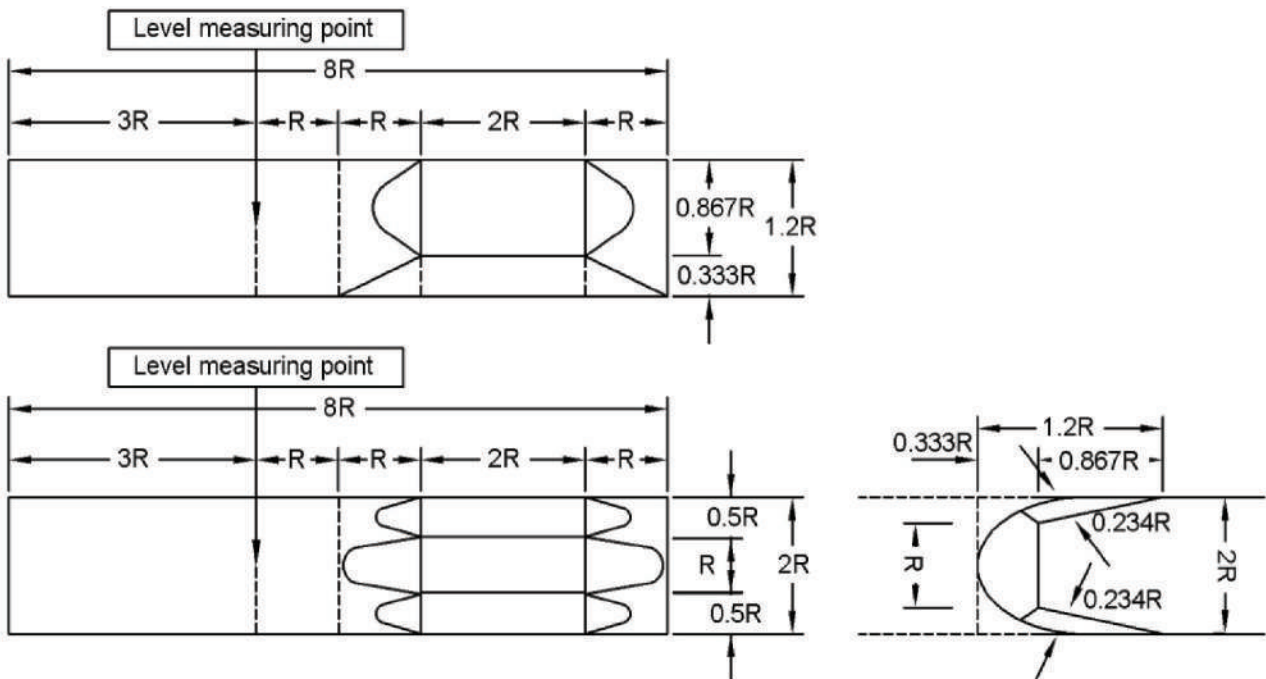


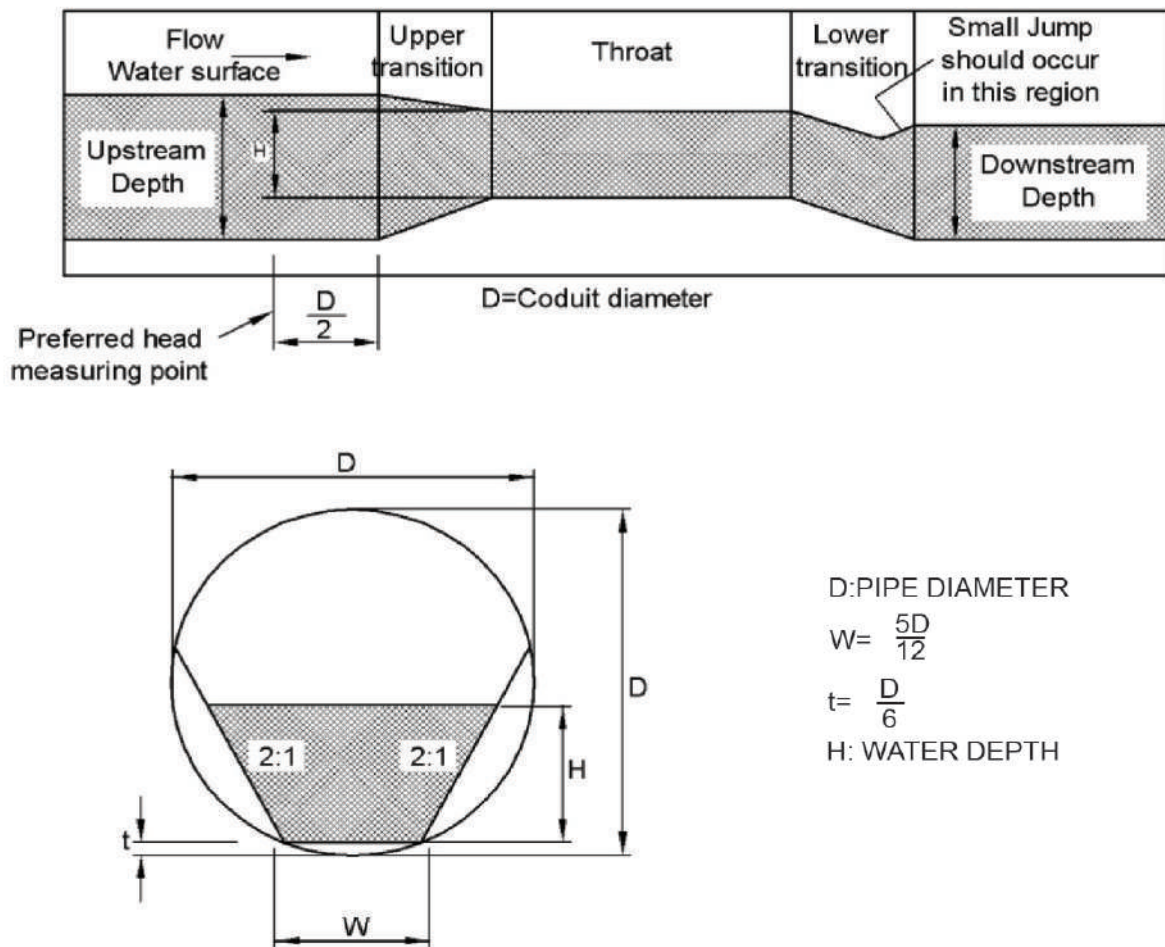Figure 3.4  Palmer-Bowlus flume installation in drains

Figure 3.5  Palmer-Bowlus flume installation in circular sewer

This has the specific advantage of its ability to be placed in a manhole to measure the sewage flow in the gravity sewer as long as the flow is not exceeding the diameter of the sewer. Typical installation details are seen in Figure 3.6 overleaf.

The depth of flow needs to be measured in only one location and thus it is a lot easier. In addition, it can be easily removed after measurement. The only disadvantage is it cannot be used when the depth of flow exceeds the diameter of the sewer and to this extent, it may have limitations in the surcharged condition of sewers in historical cities. This also has the advantage of facilitating a flow measurement in large diameter sewers, which flow under gravity and the flume itself is much simpler as in Figure 3.7.

The chart for getting at the flow once the depth is measured is obtained by relating to a standard curve supplied by the plume manufacturer depending on the shape of the plume. This is also available as software linked to a personal computer.

The combination of the Palmer-Bowlus and Tracer dye techniques have been reported as early as 1974 as illustrated in Figure 3.7. It is a system worth inducting in large trunk sewers near the outfalls to have an integrated measurement of the flows and key quality parameters or at least for the flow details and variation patterns.

Top Left and Right- The installation in manholes by inserting the pipe ends into the sewer and measuring the depth of flow by ultrasonic sensor to integrate to a computer as needed.

Bottom left- The Flume, originally invented by Palmer & Bowlus for the Los Angeles County Sanitation District and in use for over three decades, is made by many manufacturers.

Bottom Right- The installation, in a large circular sewer by merely placing the readymade flume at the invert and measurement of the depth, which can be done by ultrasonic sensor.

Figure 3.6  Configurations and use of Palmer-Bowlus flume
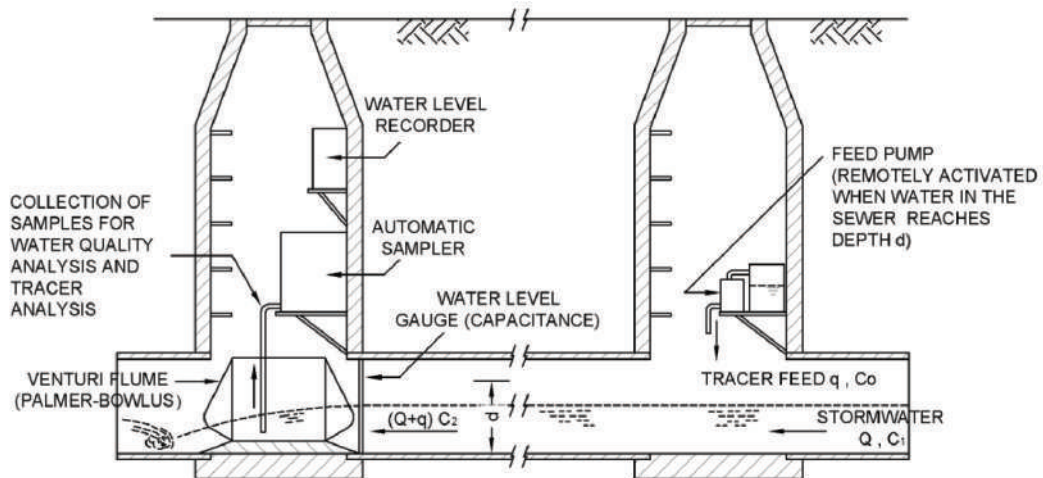


Source: J. Marsalek, 1974

Figure 3.7  Instrumentation for flow measurement and sampling in urban large conduits

f) The Venturi Pipe or the Dall Tube

While dealing with old pumping mains, there is a chance of detecting a venturi pipe fitting in the pipeline, as was the standard practice in those years. The flow through it is a function of the difference in head of the fluid at the mouth and the throat and the formula for a given venturi metre is very simple as

$$Q = K \times (a1 \times a2) \text{ (factor)} \tag{3.9}$$

$$\text{factor} = \text{SQRT } (2gh/(a1^2 - a2^2)) \tag{3.10}$$

Where
K = 0.95 to 0.98
a1 = area in sqm at mouth
a2 = area in sqm at throat
h = h1-h2
h1 = piezometric water level in m at mouth
h2 = piezometric water level in m at throat

It is thus clear that once the difference in head is measured between sewage pressure head at mouth and at throat, the square root of the same is directly proportional to the flow. It is possible to connect a differential Mercury manometer to the sampling ports in the metre and open the quarter turn-cock when flow needs to be measured and to note the reading. A simple wall chart relating the difference to the flow will be more than needed. Of course, instrumentation is possible by connecting the two pressures to a differential pressure transmitter and taking its output to a square root extractor and then to a multiplier for the constant for the metre and thereby get a continuous reading of the flow without any interventional systems.

Suffice to say that so far as estimation of flows for design of sewer systems or augmentation of sewer systems are concerned, where an existing pumping station with a venturi meter in the delivery main is available, a simple mercury manometer U tube, connected to the ports of the venturi meter may help in ascertaining the variation of the flow pattern and arrive at peak flow factors etc. more realistically.

A Dall tube is nothing but a venturi pipe-fitting of a reduced length and as otherwise all other properties of flow measurements are the same.

In fact, if possible this can be inserted into an existing pumping main for the evaluation of the above flow patterns.

## PART - 2 TYPES AND HYDRAULICS OF SEWERS

### 3.11 TYPES OF COLLECTION SYSTEM

These are separate sewers, combined sewers, pressurized sewers and vacuum sewers.

### 3.11.1 Separate Sewers

These sewers receive domestic sewage and such industrial wastes pre-treated to the discharge standards as per the Environment Protection Act 1986. The consent to discharge into sewers are given by the local pollution control administration.

### 3.11.2 Combined Sewers

These sewers receive storm water in addition and have some advantages in locations of intermittent rainfall almost throughout the year and with a terrain permitting gravitated collection and obviously being confined to a very small region as a whole. As otherwise, in regions of seasonal rainfall like in monsoons, the combined system will have serious problems in achieving self cleansing velocities during dry seasons and necessitating complicated egg shaped sewers etc. to sustain velocities at such times, plus the treatment plant to be designed to manage strong sewage in dry season and dilute sewage in monsoon season as also the hydraulics. These sewers are also ideally suited for resorts and private development.
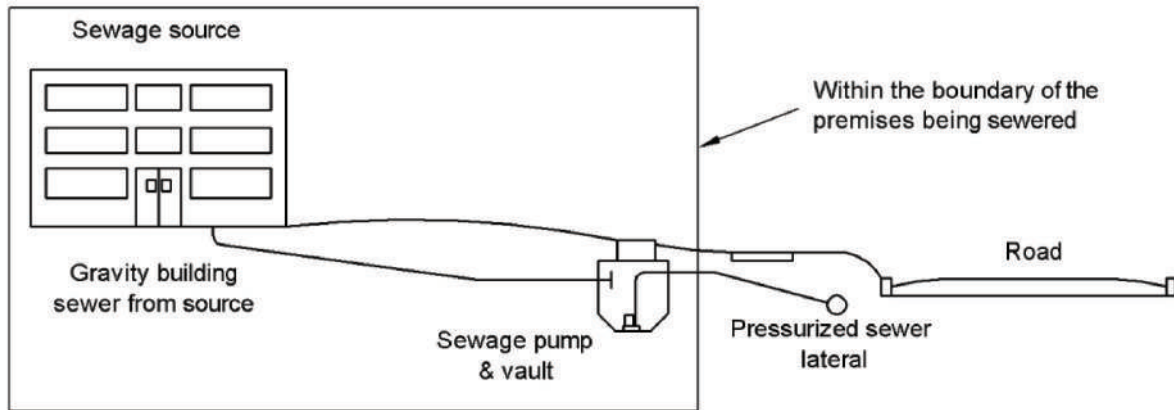
### 3.11.3 Pressurized Sewers

Pressurized sewers are for collecting sewage from multiple sources to deliver to an existing collection sewer, and/or to the STP and are not dependent on gravity and thus topography is not a challenge. Typically, sewage from establishments in the vicinity is collected in a basin fitted with submersible pump to lift and inject the sewage to a sewer on the shoulder of the roadway, thus sparing the riding surface from the infamous digging for initial repairs and often for repairs.

The principle advantages are the ability to sewer areas with undulating terrain, rocky soil conditions and high groundwater tables as pressurized sewers can be laid close to the ground and anchored well besides there cannot be infiltration, and exfiltration is quickly detected and set right and essentially smaller diameter pipes and, above all, obviating the cumbersome deep manholes as also road crossings by CI or DI pipes with trenchless technology laid inside a casing pipe and installation without disrupting traffic, opening trenches across paved roadways, or moving existing utilities etc.

An important issue is for each plot to have a grinder pump set and each commercial plot to have its own grease interceptors to remove excessive fats, oils & grease before the grinder pump. Obviously, this system is not suitable for continuous building area.

A disadvantage is the need to ensure unfailing power supply to the grinder pump and hence this is perhaps limited to high profile condominiums and not the public sewer systems in India.

A typical profile is shown in Figure 3.8 overleaf.

Source: WERF, Fact sheet 2

Figure 3.8  Profile of Pressurized Sewer system

### 3.11.4   Vacuum Sewer System

The vacuum sewer collects sewage from multiple sources and conveys it to the STP. As the name suggests, a vacuum is maintained in the collection system and when a house sewer is opened to atmospheric pressure, sewage and air are pulled into the sewer, whereby the air forms a "plug" in the line, and air pressure pushes the sewage toward the vacuum station. This differential pressure comes from a central vacuum station. These sewers can take advantage of available slope in the terrain, but have a limited capacity to pull water uphill may be to some 9 m. Each valve pit is fitted with a pneumatic pressure-controlled vacuum valve, which automatically opens after a predetermined volume of sewage has entered the sump. The difference in pressure between the valve pit (at atmospheric pressure) and the main vacuum line (under negative pressure) pulls sewage and air through the service line. The amount of air that enters with the sewage is controlled by the length of time that the valve remains open. When the vacuum valves closes, atmospheric pressure is restored inside the valve pit. Overall, the lines are installed in a saw-tooth or vertical zig-zag configuration so that the vacuum created at the central station is maintained throughout the network. A disadvantage is the need to ensure unfailing power supply to the vacuum pump and hence this is perhaps limited to high profile condominiums and not the public sewer systems in India. A typical profile is shown in Figure 3.9.
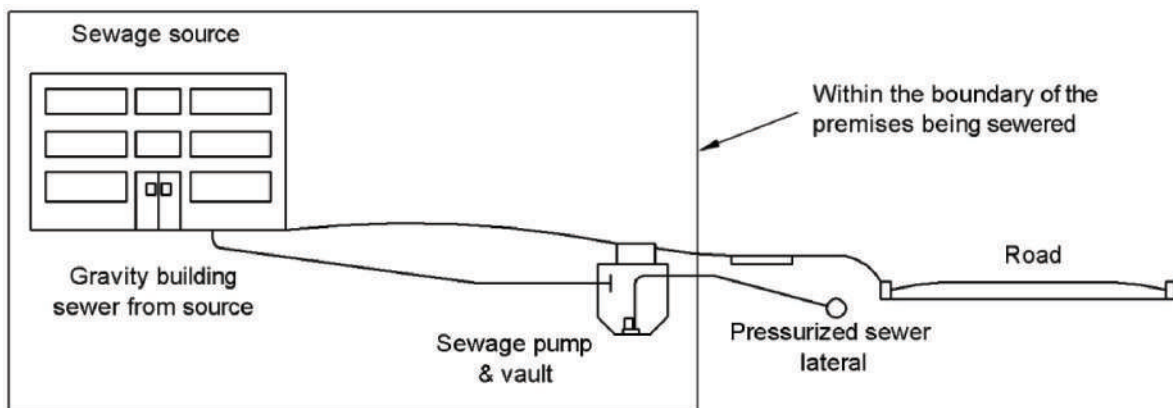


WERF, Fact sheet 4

Figure 3.9 Profile of Vacuum sewer system

## 3.12    MATERIALS, SHAPES AND SIZES OF SEWERS

### 3.12.1    Introduction

Factors influencing the selection of materials for sewers are flow characteristics, availability in the sizes required including fittings and ease of handling and installation, water tightness and simplicity of assembly, physical strength, resistance to acids, alkalies, gases, solvents, etc., resistance to scour, durability and cost including handling and installation. No single material will meet all the conditions that may be encountered in sewer design. Selection should be made for the particular application and different materials may be selected for parts of a single project. The determination of the suitability in all respects of the pipes and specials for any work is a matter of decision by the engineer concerned on the basis of requirements for the scheme and guided by Appendix A.3-10 on relative limitations on use of pipe materials in specific locations.

### 3.12.2    Brick

Brickwork is used for construction of sewers, particularly in larger diameters. Many old brick sewers are still in use and the failures are mainly due to the disintegration of the bricks or the mortar joints. Because of the comparatively higher cost, larger space requirement, slower progress of work and other factors, brick is now used for sewer construction only in special cases. The advantage of brick sewers is that these could be constructed to any required shape and size. Brick sewers shall have cement concrete or stone for invert and 12.5 mm thick cement plaster with neat finish for the remaining surface. To prevent ground water infiltration, it is desirable to plaster the outside surface. Inside plaster can be with mortar using high alumina cement conforming to IS 6452 or polyurea coating and the outer surface shall be plastered with mortar using sulphate resistant cement.

### 3.12.3    Concrete

The advantages of concrete pipes are the relative ease with which the required strength may be provided, feasibility of adopting a wide range of pipe sizes and the rapidity with which the trench may be backfilled. However, these pipes are subject to crown corrosion by sulphide gas, mid depth water line corrosion by sulphate and outside deterioration by sulphate from soil water. These shall be manufactured with sulphate resistant cement and with high alumina coating on the inside at the manufacturers works itself. Protective measures as outlined in corrosion protection in sewers shall be provided where excessive corrosion is likely to occur.

#### 3.12.3.1    Precast concrete

Plain cement concrete pipes are used in sewer systems on a limited scale only and generally, reinforced concrete pipes are used. Non-pressure pipes are used for gravity flow and pressure pipes are used for force mains, submerged outfalls, inverted siphons and for gravity sewers where absolute water-tight joints are required. Non-pressure pipes used for construction of sewers and culverts shall confirm to the IS 458. Certain heavy-duty pipes that are not specified in IS 458 should conform to other approved standards.

### 3.12.3.2 Cast-in-situ Reinforced Concrete

Cast-in-situ reinforced concrete sewers are constructed where they are more economical, or when non-standard sections are required, or when a special shape is required or when the headroom and working space are limited. The sewer shape shall be of an economic design, easy to construct and maintain and shall have good hydraulic characteristics. Wide flat culvert bottoms shall be provided with "Vee" of at least 15 cm cuvettes in the centre. All formwork for concrete sewers shall be unyielding and tight and shall produce a smooth sewer interior. Collapsible steel forms will produce the desirable sewer surface, and may be used when the sewer size and length justify the expense. It is desirable to specify a minimum clear cover of 50 mm over reinforcement steel and a minimum slump consistent with workability shall be used for obtaining a dense concrete structure free of voids. The distance for cutting concrete shall be kept to a minimum to avoid segregation and the vibrating of concrete done by approved mechanical vibrators. Air entraining cement or plasticizing agents may be used to improve workability and ensure a denser concrete. Concrete shall conform to IS 456.

### 3.12.4 Stoneware or Vitrified Clay

These pipes are normally available in lengths of 90 cm and the joints need caulking with yarn soaked in cement mortar and packing in the spigot and socket joints, which requires skilled labour. Specifications for the AA class and A class are identical except that in the case of class AA pipes, one hundred percent hydraulic testing has to be carried out at the manufacturing stage, whereas in the case of Class A only five percent of the pipes are tested hydraulically by following IS 651. The resistance of vitrified clay pipes to corrosion from most acids and to erosion due to grit and high velocities gives it an advantage over other pipe materials in handling acid concentrations. A minimum crushing strength of 1,600 kg/m is usually adopted for all sizes manufactured presently. The strength of vitrified clay pipes often necessitates special bedding or concrete cradling to improve field supportive strength.

### 3.12.5 Asbestos Cement

For sewerage works, asbestos cement pipes are usually used in sizes ranging from 80 mm to 1000 mm in diameter. Standard specifications have been framed by the BIS in IS 6908. Non corrosiveness to most natural soil conditions, freedom from electrolytic corrosion, good flow characteristics, light weight, ease in cutting, drilling, threading and fitting with specials, allowance of greater deflection up to 12 degrees with mechanical joints, ease of handling, tight joints and quick laying and backfilling are to be considered. These pipes cannot however stand high super imposed loads and may be broken easily. They are subject to corrosion by acids, highly septic sewage and by highly acidic or high sulphate soils. Protective measures as outlined in corrosion protection in sewers shall be provided in such cases. While using AC pipes strict enforcement of approved bedding-practices will reduce possibility of flexible failure.

Where grit is present, high velocities such as those encountered on steep grades may cause erosion. It is stated that in a recent process of manufacture titled Maaza, high forming pressures of up to 80 kg / sqcm, leading to very smooth surface and very few air pores are possible. However, the relevant BIS standard or code of practice is awaited.

### 3.12.6　Cast Iron

Cast Iron pipes and fittings are being manufactured in the country for several years. These pipes are available in diameters from 80 mm to 1050 mm and are covered with protective coatings. Pipes are supplied in 3.66 m and 5.5 m lengths and a variety of joints are available including socket, spigot, and flanged joints. These pipes have been classified as LA, A and B according to their thickness. Class LA pipes have been taken as the basis for evolving the series of pipes. Class A pipes allow 10 % increase in thickness over class LA. Class B pipes allows 20 % increase in thickness over class LA. Cast iron pipes with a variety of jointing methods are used for pressure sewers, sewers above ground surface, submerged outfalls, piping in sewage treatment plants and occasionally on gravity sewers where absolutely water-tight joints are essential or where special considerations require their use. IS 1536 and IS 1537 give the specifications for spun, and vertically cast pipes respectively. The advantage of cast iron pipes are long laying lengths with tight joints, ability when properly designed to withstand relatively high internal pressure and external loads and corrosion resistance in most natural soils. They are however subject to corrosion by acids or highly septic sewage and acidic soils. Whenever it is necessary to deflect pipes from a straight line either in the horizontal or in the vertical plane, the amount of deflection allowed should not normally exceed 2.5 degrees for lead caulked joints. In mechanical joints, the deflection shall be limited to 5 degrees for 80 to 300 mm dia, 4 degrees for 350 to 400 mm diameter and 3 degrees from 400 to 750 mm diameter pipes. Inside coating shall be by Cement mortar and outer coating shall be coal tar both carried out at the manufacturer's works and conforming to the relevant BIS standards/codes of practice.

### 3.12.7　Steel

Pressure sewer mains, under water river crossings, bridge crossings, necessary connections for pumping stations, self-supporting spans, railway crossing and penstocks are some of the situations where steel pipes are preferred. Steel pipes can withstand internal pressure, impact load and vibrations much better than CI pipe. They are more ductile and withstand water hammer better. For buried sewers, spirally welded pipes are relatively stronger than horizontally welded sewers. The disadvantage of steel pipe is that it cannot withstand high external load. Further, the main is likely to collapse when it is subjected to negative pressure. Steel pipes are susceptible to various types of corrosion. A thorough soil survey is necessary all along the alignment where steel pipes are proposed. Steel pipes shall be coated inside by high alumina cement mortar or polyurea and outside by epoxy. Steel pipes shall conform to IS 3589. Electrically welded steel pipes of 200 mm to 2,000 mm diameter for gas, water and sewage and laying should conform to IS 5822.

### 3.12.8　Ductile Iron Pipes

Ductile iron is made by a metallurgical process, which involves the addition of magnesium into molten iron of low sulphur content. The magnesium causes the graphite in the iron to precipitate in the form of microscopic (6.25 micron) spheres rather than flakes found in ordinary cast iron. The spheroidal graphite in iron improves the properties of ductile iron. The ductile iron pipes are normally prepared using the centrifugal cast process. The ductile iron pipes are usually provided with cement mortar lining at the factory by centrifugal process to ensure a uniform thickness throughout its length.

Cement mortar lining is superior to bituminous lining as the former provides a smooth surface and prevents tuberculation by creating a high pH at the pipe wall and ultimately by providing a physical and chemical barrier to the water.

The Indian standard IS 8329 provides specification for the centrifugally cast ductile iron pipes (similar to ISO 2531 and EN 1994). These pipes are available in the range of 80 mm to 1000 mm diameter, in lengths of 5.5 to 6 m. These pipes are manufactured in the country with ISO 9002 accreditation.

The ductile iron pipes have excellent properties of machinability, impact resistance, high wear and tear resistance, high tensile strength and ductility and corrosion resistance. DI pipes, having same composition of CI pipe, will have same expected life as that of CI pipes. They are strong, both inner and outer surfaces are smooth, free from lumps, cracks, blisters and scars. The ductile iron pipes stand up to hydraulic pressure tests as required by service regulations. These pipes are approximately 30 % lighter than conventional cast iron pipes. The ductile iron pipes are lined with cement mortar in the factory by centrifugal process and unlined ductile iron pipes are also available. The ductile iron fittings are manufactured conforming to IS 9523. The joints for ductile iron pipes are suitable for use of rubber gaskets conforming to IS 5383.

### 3.12.9 Non-Metallic Non-Concrete Synthetic Material Pipes

The main advantage of these pipes is their ability not to be affected by corrosion from sulphides or sulphates but they require precautions as detailed in clause under the sub title "laying of sewers and need to be ascertained and sorted out on specific cases. They require precautions as detailed in clause under the sub title "laying of sewers" and evaluated on case-by-case basis. An additional criterion is the ability of these pipes to withstand the mechanical jet rodding machines as in Figure 3.10 to clear the obstructions in sewers.



The mechanized reverse jet at the nozzle releasing a "jack hammer" action.

Figure 3.10  Jet rodding cum vacuum, suction sewer cleaning machine

The jet in these machines is a "jack hammer" action through a triplex plunger pump releasing the treated sewage backwards as in Figure 3.10 and the hydraulic pressures are in the range of 50 to 60 bar. The jet in the reverse direction of sewer flow acts like an airplane jet and propels the nozzle forward, and thus drills through the choked up blocks and clears the obstructions however tough they are. While the ability of the metallic and concrete sewer pipes to withstand this jet action at that pressure is by now well established in the country mainly due to the rubber ring joints, the ability of the non-metallic synthetic material sewer pipes are to be established hereafter in the country.

Moreover, their track record in other locations in such applications shall be suitably evaluated before adoption. In general, the homogenous wall composition can be relied better than multi-layered adhesive based wall composition.

### 3.12.9.1    UPVC Pipe

The chief advantages of UPVC pipe are resistance to corrosion, light weight for transportation, toughness, rigidity, economical in laying, jointing, and maintenance and easy to fabricate.

To prevent buoyancy the pipes can be tied to poles driven into the ground. IS 15328 deals with non-pressure unplasticized polyvinylchloride (PVC) for use in underground sewerage system. IS 9271 deals with the unplasticized polyvinyl chloride (UPVC) single wall corrugated pipes for drainage.

### 3.12.9.2    High Density Polyethylene (HDPE) Pipes

The advantages of these pipes offering smooth interior surfaces and offering relatively highest resistance to corrosion are recognized and they are available in solid wall. When laid in straight gradients without humps or depressions, they can easily offer longer life cycle.

Methods of joints are usually fusion welded or flange jointed depending on straight runs or fittings. Standard specifications have been framed by the BIS in IS 14333 for sewerage application.

### 3.12.9.3    Structured Wall Piping

These pipes can be manufactured in PVC-U, PP and PE as per EN 13476-3 / IS 16098. The walls of these pipes are either double walled or ribbed wall. The BIS for pipes and fittings with PVC-U material having smooth external surface Type A is IS 16098 (Part-1) and for pipes and fittings with PE and PP material having non-smooth external surface Type B is IS 16098 (Part-2). The Type B pipes are generally known as Double Walled Corrugated (DWC) pipes. In India, DWC pipes are produced in sizes 75 mm ID to 1,000 mm ID with a standard length of 6 m for easy transportation and handling and to reduce the number of joints required.

### 3.12.9.4    Glass Fibre Reinforced Plastic Pipes (GRP)

GRP Pipes are widely used in other countries where corrosion resistant pipes are required at reasonable costs. GRP can be used as a lining material for conventional pipes which are subject to corrosion. Fibre glass can resist external and internal corrosion whether the corrosion mechanism is galvanic or chemical in nature. Standard specifications have been framed by the BIS in IS 14402.

### 3.12.9.5    Fibre Glass Reinforced Plastic Pipes (FRP)

Fibre-glass reinforced plastic pipe is a matrix or composite of glass fibre, polyester resin and filters. These pipes possess better strength, durability, high tensile strength, low density and are highly corrosion resistant. Fibre-glass pressure pipes are manufactured in diameters up to 2,400 mm and length up to 18 m. These pipes are now being taken up for manufacture in India.

### 3.12.9.6    Pitch Fibre Pipes

The pitch impregnated fibre pipes are light in weight and have shown their durability in service. The pipes can be easily jointed in any weather condition as internally tapered couplings join the pipes without the use of jointing compound.

They are flexible, resistant to heat, freezing and thawing and earth currents, which cause electrolytic action. They are also unaffected by acids and other chemicals, water softeners, sewer gases, oils and greases and laundry detergents.

They can be cut to required length on the site. Due to its longer length, the cost of jointing, handling and laying is reduced. These are generally recommended for uses such as septic tanks and house connection to sewers, farm drainage, down pipes, storm drains, industrial waste drainage, etc.

These are manufactured in India with 50 to 225 mm nominal diameter and length varying from 1.5 to 3.5 m. These pipes are joined by taper coupling joints or rubber ring joints. The details of the pipes, fittings, etc., are covered in IS 11925.

## 3.13    SHAPE AND SIZE OF SEWERS

a) In general, circular sewer sections are ideal from load bearing point of view in public roads and as the hydraulic properties are better for varying flows.

b) However, for large flows, the egg-shaped sections are superior for both load transmission and velocity at minimum flows plus ability to flush out sediments in the bottom V portion when peak flow arises. These are normally of RCC, either cast in situ or pre-cast as also brickwork, though brickwork has its challenges of quality assessment and quality control.

c) Box conduits are also possible provided the inner corners are chamfered and the bottom finished as cuvettes instead of flat floor. They are perhaps best suited as a cover for taking higher diameter gravity circular sewers across roads, railway crossings, river crossings, etc. These can be built in situ with brickwork or cast in situ concrete. They can also be made in pre-cast sections duly jointed. In all cases plastering is needed on the inside and soil side and on the top side and the corners shall be filleted.

d) In early stages of new housing plot layouts, it is invariably the septic tank that is provided in the built up plots and either the septage is either sucked out periodically and sometimes surreptitiously emptied at random locations or simply discharged into the road drains or officially discharged into treatment plants or pumping stations. Nevertheless, there are many places where it is merely let into roadside drains or merely on road sides which complicates environmental issues. The twin drain system can be used, which comprises an integrally built twin drain with the drain nearer to the property carrying the septic tank effluent and grey water and the drain on the road-side carrying the storm water. The sewer drains are interconnected to flow out to treatment. A typical system in use in coastal areas of Tamil Nadu in Tsunami affected rehabilitation centres is pictured in Figure 3.11 overleaf.

<table>
<tr><td colspan="2"></td><td>The twin drain under construction and after commissioning at the Tsunami rehabilitated fisher folk settlement. The house side drain receives septic tank effluent and is with removable cover slabs.</td></tr>
</table>

Source: M/s. Kottar Social Service Society, Nagercoil & M/s. Caritas India & M/s. Caritas Germany

Figure 3.11  Twin drain system

## 3.14    MINIMUM SIZE OF CIRCULAR SEWERS

The minimum diameter may be adopted as 200 mm for cities having present / base year population of over 1 lakh. However, depending on growth potential in certain areas even 150 mm diameter can also be considered. However, in towns having present / base year population of less than 1 lakh, the minimum diameter of 150 mm shall be adopted.

In the case of hilly locations, the minimum diameter of 150 mm shall be adopted. The house sewer connection pipe to public sewer shall be (a) minimum 100 mm or higher based on the number of houses / flats connected and (b) subject to the receiving public sewer being of higher diameter.

## 3.15    FLOW IN CIRCULAR SEWERS

If the velocity and depth of flow is the same for the length of a conduit, it is termed steady flow and as otherwise, it is non-steady flow. The hydraulic analysis of sewers is simplified by assuming steady flow conditions though the actual flow conditions are different during morning peak flows and varying flows in other parts of the 24 hours.

In the design of sanitary sewers, an attempt shall be made to obtain adequate scouring velocities at the average or at least at the maximum flow at the beginning of the design period. The flow velocity in the sewers shall be such that the suspended materials in sewage are not silted up; i.e., the velocity shall be such as to cause automatic self-cleansing effect. The generation of such a minimum self cleansing velocity in the sewer, at least once a day is important, because if depositions are takes place and is not removed, it will obstruct free flow, causing further deposition and finally leading to the complete blocking of the sewer.

The smooth interior surface of a sewer pipe gets scoured due to continuous abrasion caused by the suspended solids present in sewage. It is, therefore, necessary to limit the maximum velocity in the sewer pipe. This limiting or non-scouring velocity will mainly depend upon the material of the sewer.

Thus the sewers are designed on the assumption that although silting might occur at minimum flow, it would be flushed out during peak flows. Erosion of sewers is caused by sand and other gritty material in the sewer and by excessive velocity.

### 3.15.1 Minimum Velocity for Preventing Sedimentation

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

$$V = \frac{1}{n}\left( R^{\frac{1}{6}} \sqrt{K_S (S_S - 1) d_P} \right)$$ (3.11)

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 about 0.8 for adequate self cleansing of sewers

$S_S$ = Specific gravity of particle

dp = Particle size in mm

The above formula indicates that velocity required to transport material in sewers is mainly dependent on the particle size and specific gravity and slightly dependent on conduit shape and depth of flow. The specific gravity of grit is usually in the range of 2.4 to 2.65. Gravity sewers shall be designed for the velocities as in Table 3.9.

Table 3.9  Design velocities to be ensured in gravity sewers

| No | Criteria | Value |
|----|----------|-------|
| 1 | Minimum velocity at initial peak flow | 0.6 m/s |
| 2 | Minimum velocity at ultimate peak flow | 0.8 m/s |
| 3 | Maximum velocity | 3 m/s |

Source: WPCF, ASCE, 1982

### 3.15.2 Minimum Velocity for Preventing Hydrogen Sulphide in Sewers

The velocity shall be not only self cleansing but also be sufficient to keep the submerged surfaces of the sewer free from slimes and prevent the generation of Hydrogen Sulphide gas which can attack the cement concrete sewers. It is useful to define a climatic condition as the combination of the average temperature for the warmest three months of the year and the average 6-hour high flow BOD for the day. Where diurnal BOD curves have not been made, it may be assumed that this BOD is 1.25 times the BOD of flow proportioned 24-hour composite. The effective BOD defined by the equation:

$$(EBOD) = (BOD)_c \times 1.07^{(Tc-20)}$$ (3.12)

Where

EBOD = Effective EBOD in mg/l

$(BOD)_c$ = Climatic BOD in mg/l

$T_c$ = Climatic temperature in degrees Celsius

1.07 = Empirical coefficient

The reference for E BOD is from ASCE Manuals and Reports on Engineering Practice No. 60.

### 3.15.2.1    Potential for Sulphide Build up

Another indicator of the likelihood of sulphide build up in relatively small gravity sewers (not over 600 mm diameter) is given by the formula:

$$Z = [EBOD/(S^{0.50} \times Q^{0.33})] \times (P/b) \tag{3.13}$$

Where

|  |  |  |
|---|---|---|
| Z = | Defined function |
| S = | Hydraulic slope |
| Q = | Discharge volume in m3/sec |
| P = | Wetted perimeter in meters |
| b = | Surface width in meters. |

The reference for sulphide generation is WPCF, ASCE, 1982.

The sulphide generation based on Z values are given in Table 3.10

Table 3.10  Sulphide generation based on Z values

| Z Values | Sulphide Condition |
|---|---|
| Z < 5,000 | Sulphide rarely generated |
| 5,000 ≤ Z ≤ 10,000 | Marginal condition for sulphide generation |
| Z > 10,000 | Sulphide generation common |

Source: WPCF, ASCE, 1982

### 3.15.3    Maximum Velocity

Erosion is caused by sand and other gritty material and is compounded by high velocities and hence the maximum velocity shall be limited to 3 m/s. In hilly areas where the slope and flows gets fixed, the velocity also gets automatically fixed. If such velocities exceed 3 m/sec in hilly areas, use of cast iron and ductile iron pipes shall be made with socket and spigot joints and O rings and the sockets facing uphill. The provision of structures like drop manholes can also be made to dissipate the energy.

### 3.15.4    Manning's Formula for Gravity Flow

$$V = [(1/n)] \times [R^{2/3} S^{1/2}] \tag{3.14}$$

For circular conduits

$$V = (1/n)(3.968 \times 10^{-3}) D^{2/3} S^{1/2} \tag{3.15}$$

and

$$Q = (1/n)(3.118 \times 10^{-6}) D^{2.67} S^{1/2} \tag{3.16}$$

where,

$Q$ : Discharge in l/s

$S$ : Slope of hydraulic gradient

$D$ : Internal diameter of pipe line in mm

$R$ : Hydraulic radius in m

$V$ : Velocity in m/s

$n$ : Manning's coefficient of roughness as in Table 3-11

A chart for Manning's formula is in Appendix A.3.2 A and A 3.2 B for the stated ranges of discharges therein. These can be used to initially verify

(a) the tentative size, and slope of the required sewer for a given flow rate and velocity, or

(b) The tentative flow rate and slope of a chosen sewer size and velocity.

It is not easy to read these values precisely to decimal values from the graph and hence, it is recommended to recheck the values in the MS Excel spreadsheet given in Appendix A.3.3. There are also similar nomograms, etc. but the precision is best obtained in MS Excel.

### 3.15.5 Design Depth of Flow

The sewers shall not run full as otherwise the pressure will rise above or fall below the atmospheric pressure and condition of open channel flow will cease to exist. Moreover, from consideration of ventilation, sewers should not be designed to run full. In case of circular sewers, the Manning's formula reveals that:

- The velocity at 0.8 depth of flow is 1.14 times the velocity at full depth of flow.

- The discharge at 0.8 depth of flow is 0.98 times the discharge at full depth of flow.

Accordingly, the maximum depth of flow in design shall be limited to 0.8 of the diameter at ultimate peak flow. In order to facilitate the calculations easily, the hydraulic properties at various depths of flow are compiled in Figure 3.12 and Figure 3.13 and Table 3.12.

Table 3.11  Manning's coefficient of roughness n for stated materials

| Type of Material | | Condition | Manning's n |
|---|---|---|---|
| Salt glazed 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 | (a) Neat cement plaster | | 0.018 |
| | (b) Sand and cement plaster | | 0.015 |
| | (c) Concrete, steel troweled | | 0.014 |
| | (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 |
| Stone-work | (a) Smooth, dressed ashlar | | 0.015 |
| | (b) Rubble set in cement | | 0.017 |
| | (c) Fine, well packed gravel | | 0.020 |
| Earth | (a) Regular surface in good condition | | 0.020 |
| | (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 |
| Steel | (a) Welded | | 0.013 |
| | (b) Riveted | | 0.017 |
| | (c) Slightly tuberculated | | 0.020 |
| | (d) With spun cement mortar lining | | 0.011 |
| Cast Iron / Ductile Iron | (a) Unlined | | 0.013 |
| | (b) With spun cement mortar lining | | 0.011 |
| Asbestos cement | | | 0.011 |
| 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.

Source: CPHEEO, 1999
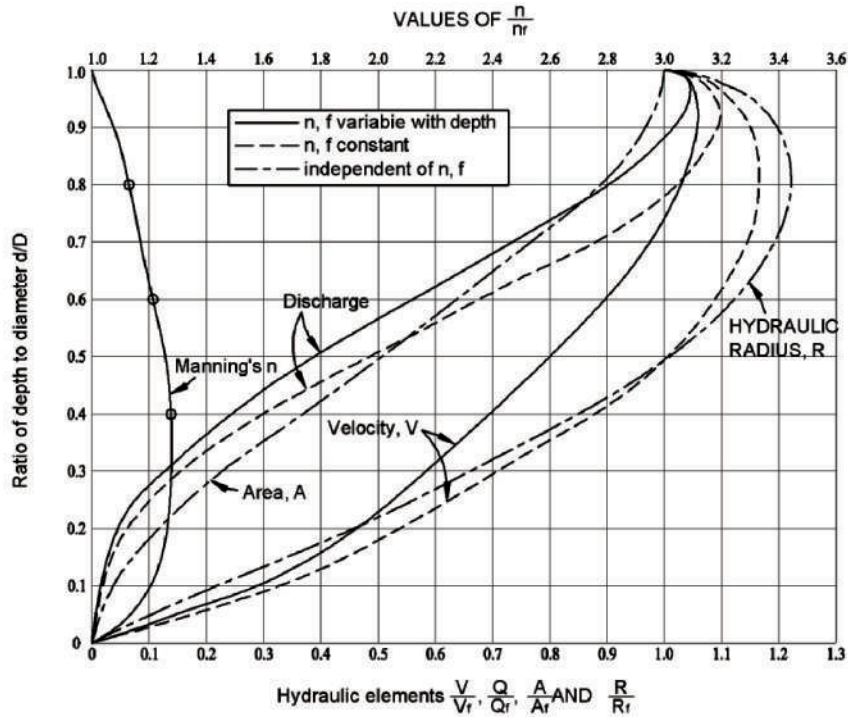
Source: CPHEEO, 1993

Figure 3.12  Hydraulic – Element graph for circular sewers
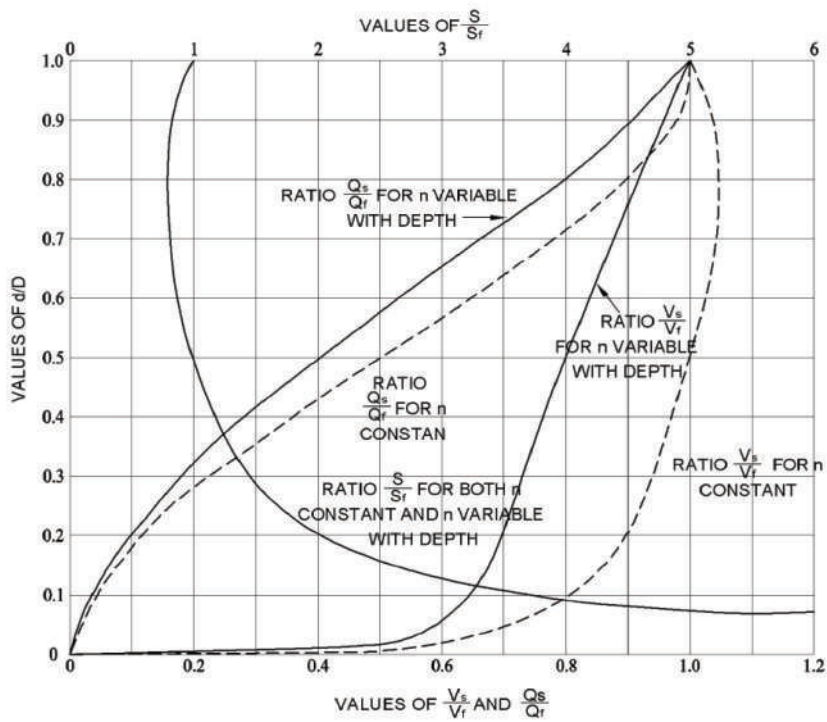


Source: CPHEEO, 1993

Figure 3.13  Hydraulic elements of circular sewers that possess equal self-cleansing
properties at all depths

Table 3.12  Hydraulic properties of circular sections for Manning's formula

| Constant (n) | | | Variable (n) | | |
|---|---|---|---|---|---|
| d/D | v/V | q/Q | $n_d$/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 | 0712 |
| 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 |

Source: CPHEEO, 1993

Where,

| | |
|---|---|
| D = Depth of flow (internal dia) | d = Actual depth of flow |
| V = Velocity at full depth | v = Velocity at depth 'd' |
| n = Manning's coefficient at full depth | nd = Manning's coefficient at depth 'd' |
| Q = Discharge at full depth | q = Discharge at depth 'd' |

For practical purposes, it is not possible to measure the value of $n_d$ and hence only the fixed value of Manning's n shall be used. The method of using the Table 3-12 is illustrated in Appendix A.3.4 A and A 3.4 B for stated ranges of discharges for typical cases in day-to-day situations of design of circular sewer pipes under gravity flow conditions. In as much as the determination of the varying values of n is difficult and has many uncertainties, the formula shall be used with constant values of n only.

### 3.15.6   Slope of Sewer

The minimum slopes in Table 3.13 shall be applicable:

Table 3.13  Minimum slopes of sanitary sewers

| Sewer Size (mm) | Minimum Slope | | Sewer Size (mm) | Minimum Slope | |
|---|---|---|---|---|---|
| | As percent | As 1 in | | As percent | As 1 in |
| 150 | 0.6 | 170 | 375 | 0.15 | 670 |
| 200 | 0.40 | 250 | 450 | 0.12 | 830 |
| 250 | 0.28 | 360 | ≥525 | 0.10 | 1000 |
| 300 | 0.22 | 450 | | | |

## 3.16 HYDRAULICS OF SEWERS FLOWING UNDER PRESSURE

### 3.16.1 Type of Flow

The hydraulic analysis of pumping mains is approached based on turbulent flow conditions to ensure that the suspended matter does not settle during pumping.

### 3.16.2 Hazen-Williams Formula

$$V = 0.849 \, C \, R^{0.63} \, S^{0.54} \tag{3.17}$$

for circular conduits, the expression becomes

$$V = 4.567 \times 10^{-3} \, C \, D^{0.63} \, S^{0.54} \tag{3.18}$$

and

$$Q = 1.292 \times 10^{-5} C \, D^{2.63} \, S^{0.54} \tag{3.19}$$

where,

$Q$ : Discharge in m³/hr

$D$ : Internal diameter of pipe in mm

$V$ : Velocity in m/s

$R$ : Hydraulic radius in m

$S$ : Slope of hydraulic gradient and

$C$ : Hazen – Williams coefficient as in Table 3.14 overleaf.

A chart for Hazen William's formula is in Appendix A. 3.5 A and A 3.5 B for stated ranges of discharges. This can be used to initially verify -:

(a) The tentative size, and slope of the required sewer for a given flow rate and velocity, or

(b) The tentative flow rate and slope of a chosen sewer size and velocity.

It is not easy to read these values precisely to decimal values from the graph and hence, it is recommended to recheck the values in the MS Excel given in Appendix A.3.6.

## 3.17 SEWER TRANSITIONS

### 3.17.1 Connections of Different Sewers

Where sewers of different characteristics are connected, sewer transitions occur. The difference may be in terms of flow, area, shape, grade, alignment and conduit material, with a combination of one or all characteristics. Transitions may be in the normal cases streamlined and gradual and can occur suddenly in limiting cases. Head lost in a transition is a function of velocity head and hence assumes importance in the flat terrain. Deposits also impose significant losses. For design purposes, it is assumed that energy losses and changes in depth, velocity and invert elevation occur at the centre of transition and afterwards these changes are distributed throughout the length of transition.

Table 3.14  Hazen-Williams coefficients

| No. | Conduit Material | Recommended values for | |
|---|---|---|---|
| | | New Pipes(A) | Design |
| 1 | Unlined metallic pipes | | |
| | Cast iron, Ductile iron | 130 | 100 |
| | Mild steel | 140 | 100 |
| 2 | Centrifugally lined metallic pipes | | |
| | Cast iron, Ductile iron and Mild steel pipes lined with cement mortar or epoxy | | |
| | Up to 1200 mm dia | 140 | 140 |
| | Above 1200 mm dia | 145 | 145 |
| 3 | Projection method cement mortar lined metallic pipes | | |
| | Cast iron, Ductile iron and Mild steel pipes | 130(B) | 110(C) |
| 4 | Non-metallic pipes | | |
| | RCC spun concrete upto 1200 mm diameter | 140 | 140 |
| | RCC spun concrete above 1200 mm diameter | 145 | 145 |
| | Pre-stressed concrete upto 1200 mm diameter | 140 | 140 |
| | Pre-stressed concrete above 1200 mm diameter | 145 | 145 |
| | Asbestos cement | 150 | 140 |
| | HDPE, UPVC, GRP, FRP | 150 | 140 |

Note:

A  The C values for new pipes included in the above table are for determining the acceptability of surface finish of new pipelines. The user agency may specify that flow test may be conducted for determining the C values of laid pipelines.

B  For pipes of diameter 500 mm and above, the range of C values may be from 90 to 125 for pipes less than 500 mm.

C  In the absence of specific data, this value is recommended. However, in case authentic field data is available, higher rates upto 130 may be adopted.

Note: Even though the C value can be taken as 145 for Water supply, but for sewage, 140 shall be taken for design purpose.

Source: CPHEEO, 1999

The energy head, piezometric head (depth) and invert as elevation are noted and working from Energy grade line, the required invert drop or rise is determined. However, if the calculations indicate a rise in invert it is ignored since such a rise will create a damming effect leading to deposition of solids.

For open-channel transition in subcritical flow the loss of energy is expressed as:

$$Head\ Loss = K\ (V^2/2g)$$  (3.20)

Where

(V²/2g) is change of velocity head before and after transition,

K = 0.1 for contractions and 0.2 for expansions.

In transitions for supercritical flow, additional factors must be considered, since standing waves of considerable magnitude may occur or in long transitions air entrapment may cause backing of flow. Allowance for the head loss that occurs at these transitions has to be made in the design.

Manholes shall be located at all such transitions and a drop shall be provided where the sewer is intercepted at a higher elevation for streamlining the flow, taking care of the head loss and to help in maintenance. 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:

(a)    For sewers less than 400 mm          Half the difference in dia.

(b)    400 mm to 900 mm                         2/3 the difference in dia.

(c)    Above 900 mm                                4/5 the difference in dia.

Transition from larger to smaller diameters shall not be made. The crowns of sewers are always kept continuous. In no case, the hydraulic flow line in the large sewers shall be higher than the incoming one. To avoid backing up, the crown of the outgoing sewer shall not be higher than the crown of incoming sewer.

### 3.17.2   Bends

The head loss in bends is expressed by:

$$h_b = k_b\ V^2/2g$$  (3.21)

Where

$k_b$ is a friction coefficient, which is a function of the ratio of radius of curvature of the bend to the width of conduit, deflection angle, and cross section of flow.

The friction factor for various fittings are given in Table 4-2 in Chapter 4.

### 3.17.3    Junction

A junction occurs where one or more branch sewers enter a main sewer. The hydraulic design is in effect, the design of two or more transitions, one for each path of flow. Apart from hydraulic considerations, well-rounded junctions are required to prevent deposition. Because of difficulty in theoretically calculating the hydraulic losses at junctions, some general conditions may be checked to ensure the proper design of junctions. If available energy at junctions is small, gently sloping transitions may be used. The angle of entry may be 30 degrees or 45 degrees with reference to axis of main sewer, whenever ratio of branch sewer diameter to main sewer diameter is one half or less. Junctions are sized so that the velocities in the merging streams are approximately equal at maximum flow. If considerable energy is available in long sewers at a junction, a series of steps may be provided in the branch to produce a cascade or it may be designed as a hydraulic jump to dissipate energy in the branch before entering the main sewer. Vertical pipe drops are used frequently at junctions for which main sewer lies well below the branch sewers, particularly if the ratio of branch sewer diameter and main sewer diameter is small. These pipe drops are designed with an entrance angle of 30 degrees with the main sewer.

### 3.17.4    Vertical Drops and Other Energy Dissipators

In developed areas, it may sometimes be necessary and economical to take the trunk sewers deep enough like tunnels. In such cases, the interceptors and laterals may be dropped vertically through shafts to the deep trunk sewers or tunnels. Hydraulic problems encountered with such deep vertical drops may be difficult to solve and may be some times solved by model studies. Vertical drops must be designed to avoid entrapment of air. Such entrapped air in a shaft can result in surges, which may reduce the capacity of intake. Entrapped air may not be able to flow along the sewer and escape through another ventilation shaft. Air problems can be minimised by designing a shaft with an open vortex in the middle for full depth of drop. To accomplish this, the flow is to be inducted tangentially into inlet chamber at the head of the shaft. If the vertical drop is likely to cause excessive turbulence, it may be desirable to terminate the drop in the branch to dampen the flow before it enters the main flow. Another type of vertical drop incorporates a water cushion to absorb the impact of a falling jet. Water cushion required has been found to be equal to h1/2 d1/3 in which h is the height of fall and d is depth of the crest. Special chutes or steeply inclined sewers can be constructed instead of vertical drops. All drops cause release of gasses and maintenance problems, and hence shall be avoided to the extent where possible.

### 3.17.5    Inverted Siphon

When a sewer line dips below the hydraulic grade line, it is called an inverted siphon. The purpose is to carry the sewer under the obstruction and regain as much elevation as possible after the obstruction is passed. They shall be resorted to only where other means of passing the obstruction are not feasible, as they require considerable attention in maintenance. As the siphons are depressed below the hydraulic grade line, maintenance of self-cleansing velocity at all flows is very important. Two considerations that govern the profile of a siphon are provision for hydraulic losses and ease of cleaning. It is necessary to ascertain the minimum flows and the peak flows for design.

To ensure self-cleansing velocities for the wide variations in flows, generally, two or more pipes not less than 200 mm diameter are provided in parallel so that up to the average flows, the first pipe is used and when the flow exceeds the average, the balance flow is taken by the second and subsequent pipes. Siphons may need cleaning more often than gravity sewers and hence shall not have any sharp bends either horizontal or vertical. Only smooth curves of adequate radius shall be used and the entry and exit piping shall be at a slope of as close to 30 degrees to horizontal. The design criteria for inverted siphons are given in IS 4111, Part III. Some of the important criteria are given below.

### 3.17.6 Hydraulic Calculations for Inverted Siphon

As the inverted siphon is a pipe under pressure, a difference in the water levels at the inlet and outlet is the head under which the siphon operates. This head shall be sufficient to cover the entry, exit and friction losses in pipes. The friction loss through the barrel will be determined by the design velocity. The Hazen-Williams formula can be used for calculation of head loss.

### 3.17.7 Velocity in Inverted Siphon Sewers

It is necessary to have a self-cleansing velocity of 1.0 m/s for the minimum flow to avoid deposition in the line.

### 3.17.8 Size and Arrangement of Pipes

In the multiple-pipe siphons, the inlet shall be such that the pipes come into action successively as the flow increases. This may be achieved by providing lateral weirs with heights kept in accordance with the depth of flow at which one or more siphon pipes function. Figure 3.14 gives the general arrangement for a three-way siphon. In the two-pipe siphon, the first pipe shall take 1.25 to 1.5 times the average flow and second shall take the balance of the flow.
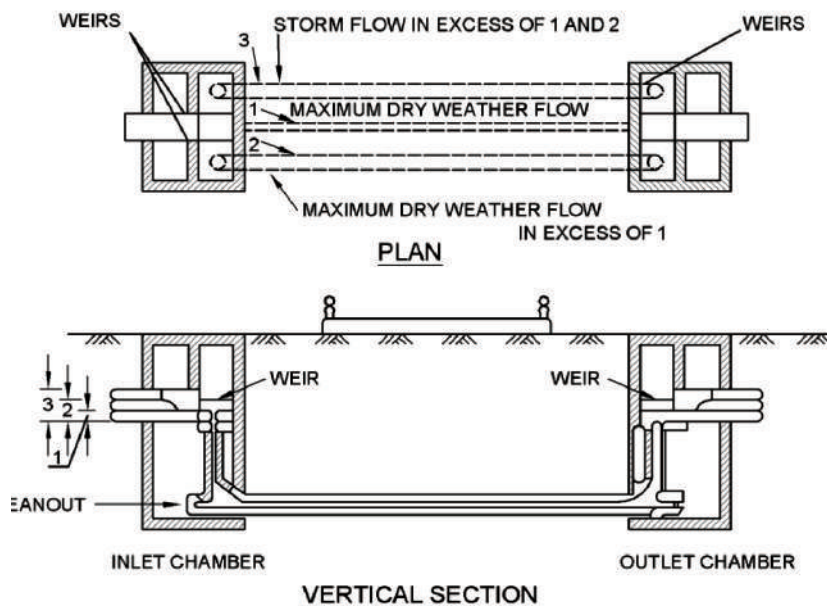


Source: CPHEEO, 1993

Figure 3.14  Inverted siphon or suppressed sewer for combined sewage

### 3.17.9    Inlet and Outlet Chambers

The design of inlet and outlet chambers shall allow sufficient room for entry for cleaning and maintenance of siphons. The outlet chambers shall be so designed as to prevent the backflow of sewage into pipes which are not being used at the time of minimum flow.

### 3.17.10    General Requirements

Provision shall be made for isolating the individual pipes as well as the siphon to facilitate cleaning. This can be done by providing suitable penstocks or stop boards at the inlet and outlet of each pipe and by providing stop valve at its lower point if it is accessible. A manhole at each end of the siphon shall be provided with clearance for rodding. The rise, out of the siphon for small pipes shall be on a moderate slope so that sand and other deposits may be moved out of the siphon. The rising leg shall not be so steep as to make it difficult to remove heavy solids by cleaning tools that operate on hydraulic principle. Further, there shall be no change of diameter in the barrel since this would hamper cleaning operation. It is desirable to provide a coarse screen to prevent the entry of rags etc. into the siphon.

Proper bypass arrangements shall be provided from the inlet chamber and if required special arrangements shall be made for pumping the sewage to the lower reach of sewer line. Alternatively, a vacuum pump may be provided at the outlet to overcome maintenance problems arising out of clogging and silting of siphons. If it is possible a blow off may be installed at the low point to facilitate emergency maintenance operations.

Positive pressure develops in the atmosphere upstream of a siphon because of the downstream movement of air induced by the sewage flow. This air tends to exhaust from the manhole at the siphon inlet. The exiting air can cause serious odour problems. Conversely, air is drawn in at the siphon outlet. Attempts can be made to close the inlet structure tightly so that the air gets out at manholes or vents upstream. However, this causes depletion of oxygen in the sewer and leads to sulphide generation. To avoid this, sufficient ventilation arrangements have to be provided.

### 3.18    LEAPING WEIRS FOR SEGREGATING STORM FLOWS

Even though sometimes sewers are required to be designed for accommodating the storm flows as a combined sewer, it may not be necessary to design the treatment plant for the full combined flow. The classic principle that storm water being lighter in density will be floating over the denser sewage is recognized to design the leaping weirs by which the lighter storm water is diverted before the treatment plant. The two variations of this facility are presented herein.

### 3.18.1    Side Flow Leaping Weirs

A side flow weir constructed along one or both sides of a combined sewer delivers excess flows during storm periods to relief sewers or natural drainage courses. The crest of the weir is set at an elevation corresponding to the desired depth of flow in the sewer. The weir length must be sufficiently long for effective regulation. The length of the side flow weir is given by the formula devised by Babbitt.

$$L = 7.6 \times 10^{-4} \times V \times D \times Log\ (h_1/h_2) \hspace{2cm} (3.22)$$

where,

$L$ :      Required length in m

$V$ :      Velocity of approach in m/s

$D$ :      Dia of the sewer in mm

$h1, h2$ :   Heads in m above the crest of the weir upstream and downstream

The formula is limited to conditions in which the weir is placed in the side of a circular pipe at a distance above the bottom, greater than d/4 and less than d/2 where 'd' is the diameter of the pipe and the edge of the weir is sharp and parallel to the invert of the channel. Its usefulness is limited in that it was devised for pipes between 450 and 600 mm in diameter and where the depth of flow above the weir should not exceed 3d/4. A typical sideways leaping weir is shown in Figure 3.15.



Incoming flows in excess of the desired dry weather flow will "leap" over the weirs on the sides to be diverted to storm drans. This can also be used for a flow equalisation to the STP. When the sewage level drops below the weir lip, submersible pumps can pump back the sewage from the bottom well so that a constant rate of flow can be maintained to the STP. The downstream screen after this unit and Parshall flume with stilling well and level metre are also seen.

Source: Daniel Sztruhar et.al.

Figure 3.15  Typical sideways leaping weir

### 3.18.2   Floor Level Leaping Weir

A floor-level leaping weir is formed by a gap in the invert of a sewer through which the dry-weather flow falls and over which a portion of the entire storm leaps. Leaping weirs have the advantage of operating as regulators without moving parts, but the disadvantage of concentrating grit in the low flow channel. Some formulae based in empirical findings are available for design. However, from practical considerations, it is desirable to design the weirs with moving crests to make the opening adjustable as indicated in Figure 3.16. A typical-floor level leaping weir is shown in Figure 3.17 overleaf.
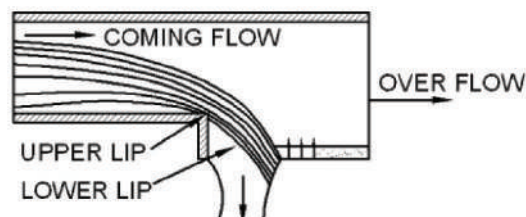


Source: CPHEEO, 1993

Figure 3.16  Leaping weir

The incoming sewage (towards the reader), will drop through the floor level weir and be taken away through the pipe to the right of the reader. During heavy inflows, the excess quantity will leap past the floor level weir and led away to either storm sewer or balancing tank in case of STP to a balancing tank below and to be pumped back during low flows for constant flow to the STP.

Source: Daniel Sztruhar et.al.

Figure 3.17  Typical floor level leaping weir

## 3.19    RELIEF SEWERS

An overloaded existing sewer may require relief, with the relief sewer constructed parallel to the existing line. Relief sewers are also called supplementary sewers. In the design, it must be decided whether (a) the proposed sewer is to share all the rates of flow with the existing sewer or (b) it is to take all flows in excess of predetermined quantity or (c) it is to divert a predetermined flow from the upper end of the system.

The topography and available head may dictate which alternative is selected. If flows are to be divided according to a ratio, the inlet structure to the relief sewer must be designed to divide the flow. If the relief sewer is to take all flows in excess of a predetermined quantity, the excess flow may be discharged through a weir to the relief sewer. If the flow is to be diverted in the upper reaches of a system, the entire flow at the point of diversion may be sent to the relief sewer or the flow may be divided in a diversion structure.

A decision as to the method of relief to be chosen depends on available velocities. Self-cleansing velocities have to be maintained in both sewers even after diversion of flows. Otherwise, nuisance conditions may result. If the relief sewer is designed to take flows in excess of a fixed quantity the relief sewer itself will stand idle much of the time and deposits may occur, in some cases, it might be better to make the new sewer large enough to carry the total flow and abandon the old one.

**PART - 3 DESIGN OF SEWER NETWORK**

## 3.20    BASIC INFORMATION

Before the sewer network can be designed, accurate information regarding the site conditions is essential. This information may vary with the individual scheme but, shall in general, be covered by the following:

a) Site plan - A plan of the site to scale with topographical levels, road formation levels, level of the outfall, location of wells, underground sumps and other drinking water sources

b) The requirements of local bye-laws

c) Subsoil conditions - Subsoil conditions govern the choice of design of the sewer and the method of excavation

d) Location of other services (such as position, depth and size of all other pipes, mains, cables, or other services, in the vicinity of the proposed work)

e) Topography

### 3.20.1    Preliminary Investigation for Design of Sewer System

The anticipation of future growth in any community in terms of population or commercial and industrial expansion forms the basis for preparation of plan for providing the amenities including installation of sewers in the area to be served. The anticipated population, its density and its waste production is generally estimated for a specified planning period. The recommended planning period is 30 years; however, this may vary depending upon the local conditions. The prospective disposal sites are selected and their suitability is evaluated with regard to physical practicability for collection of sewage, effects of its disposal on surrounding environment and cost involved.

### 3.20.2    Detailed Survey

The presence of rock or underground obstacles such as existing sewers, water lines, electrical or telephone wires, tunnels, foundations, etc., have significant effect upon the cost of construction. Therefore, before selecting the final lines and grades for sewers necessary information regarding such constructions is collected from various central and state engineering departments.

Besides the location of underground structure, a detailed survey regarding the paving characteristics of the streets, the location of all existing underground structures, the location and basement elevations of all buildings, profile of all streets through which the sewer will run, elevations of all streams, culverts and ditches, and maximum water elevations therein are also made. The above details are noted on the map. The scale of the map may vary depending upon the details desired. It is recommended to adopt the following scales for various plans and drawings depending upon the detailed information desired.

(a) Index Plan     -     1 : 100,000 or 1 : 200,000

(b) Key Plan and general layout     -     1 : 10,000 or 1 : 20,000

(c) Zonal Plans     -     1 : 2,500 or 1 : 5,000

(d) Longitudinal sections of sewers -     1 : 500 or 1 : 2,250 or 1 : 2,500

(e) Structural drawings     -     1 : 20 or 1 : 50 or 1 : 100 or 1 : 200

### 3.20.3 Layout of System

The sewer system layout involves the following steps:

(a) Selection of an outlet or disposal point

(b) Prescribing limits to the drainage valley or Zonal Boundaries

(c) Location of Trunk and Main Sewers

(d) Location of Pumping Stations if found necessary

In general, the sewers will slope in the same direction as the street or ground surface and will be connected to trunk sewers. The discharge point may be a treatment plant or a pumping station or a water course, a trunk sewer or intercepting sewer. It is desirable to have discharge boundaries following the property limits. The boundaries of sub zones are based on topography, economy or other practical consideration.Trunk and main sewers are located in the valleys. The most common location of sanitary sewer is in the centre of the street. A single sewer serves both sides of the street with approximately same length for each house connection.

In very wide streets it may be economical to lay a sewer on each side. In such cases, the sewer may be adjacent to the road curb or under the footpath & interference with other utilities has to be avoided. Sometimes sewers may be located in the back of property limits to serve parallel rows of houses in residential area. However, access to such locations becomes difficult and hence sewer locations in streets are often preferred. Sewers as a rule are not located in proximity to water supplies. When such situations are unavoidable the sewers may be encased in sleeve pipes or encased in concrete.

The Puducherry Public Works Department has been historically adopting a practice of connecting the house services of a few houses by a rider sewer on the foot path with chambers and then connect to the sewer manhole in the road as in Figure 3.18 (overleaf).

A tentative layout is prepared by marking sewer lines along the streets or utilities / easements. The direction of flow is shown using arrows, which is generally the direction in which the ground slopes. Manholes are provided at all sewer intersections, changes in horizontal direction, major change in slopes, change in size and at regular intervals.

The depth of cut is dictated by the need to ensure a minimum cover and the desirability of mandatory cushion depending upon the pipe size and expected loads.

It is the standard design practice to provide a minimum cover of 1 m at the starting point in the case of sanitary sewer network and 0.5 m for storm drainage system.

Photograph on left shows connection from interceptor chamber to manhole and photo-
graph on right shows connection from chamber to manhole

Source: Puducherry PWD

Figure 3.18  House sewer connections

If the sewer changes in direction in a manhole without change of size, a drop of usually 30 mm is provided in the manhole. If the sewer changes in size, the crown of inlet and outlet sewers are set at the same elevation. The vertical drop may be provided as described in Section 3.17. Sewers as a design practice are not located in proximity to water supplies. When such situations are unavoidable, the sewers should be encased in sleeve pipes or encased in concrete. Even when a plot is empty, it is better to lay the house service connection sewer to the nearest manhole from a temporary chamber in the vacant plot and plug it.

### 3.20.4    Profile of Sewer System

The vertical profile is drawn from the survey notes for each sewer line. All longitudinal sections are indicated with reference to the same datum line. The vertical scale of the longitudinal sections are usually magnified ten times the horizontal scale. The profile shows ground surface, tentative manhole locations, grade, size and material of pipe, ground and invert levels and extent of concrete protection, etc. At each manhole the surface elevation, the elevation of sewer invert entering and leaving the manhole are generally listed.

### 3.20.5    Available Head

Generally, the total available energy is utilized to maintain proper flow velocities in the sewers with minimum head loss. However, in hilly terrain excess energy may have to be dissipated using special devices. Hence, the sewer system design is limited on one hand by hydraulic losses, which must be within the available head and on the other to maintain self-cleansing velocities. It becomes difficult to meet both conditions with increasing variation in rate of flow. Where differences in elevations are insufficient to permit gravity flow, pumping may be required. The cost of construction, operation, and maintenance of pumping stations are compared with the cost of construction and maintenance of gravity sewers. Apart from the cost considerations the consequences of mechanical and electrical failures at pumping stations may also be considered, which may necessitate a gravity system even at a higher cost.

### 3.20.6 Plans and Nomenclature

The following procedure is recommended for the nomenclature of sewers:

- First distinct number such as 1, 2, 3, etc., is allotted to the manholes of the trunk sewers commencing from the lower end (outfall end) of the line and finishing at the top end.

- Manholes on the mains or sub mains are again designated numbers 1, 2, 3, etc., prefixing the number of the manhole on trunk/main sewer where they join. Similar procedure is adopted for the branches to branch main. When all the sewer lines connected to the main line have thus been covered by giving distinctive numbers to the manholes, the manholes on the further branches to the branch mains are similarly given distinctive numbers, again commencing with the lower end.

- If two branches, one on each side meeting the main sewer or the branch sewer, letter 'L' (to represent left) or letter 'R' (to represent right) is prefixed to the numbering system, depending on the direction of flow.

- If there is more than one sewer either from the left or right they are suitably designated as $L_1$, $L_2$, $L_3$, or $R_1$, $R_2$, $R_3$, the subscript refer to the line near to the sewer taking away the discharge from the manhole.

Thus, $L_2R4.2.3$ (Figure 3-19 overleaf) will pinpoint a particular manhole on the sub main from which the flow reaches manhole number 4 on the trunk sewer through a sub main and a main. The first numeral (from the left) is the number of the manhole on the trunk sewer. The numerals on the right of this numeral, in order, represent the manhole numbers in the main, sub main, etc., respectively.

The first letter immediately preceding the numeral denotes the main and that it is to the right of the trunk sewer. Letters to the left in their order represent sub main, branch respectively. The same nomenclature is used for representing the sections, e.g. Section $L_2R4.2.3$ identifies the section between the manhole $L_2R4.2.3$ and the adjoining downstream manhole.

All longitudinal sections should be indicated with reference to the same datum line. The vertical scale of the longitudinal sections should be magnified ten times the horizontal scale.

The trunk sewer should be selected first and drawn and other sewers should be considered as branches. The trunk sewer should be the one with the largest diameter that would extend farthest from the outfall works. Whenever two sewers meet at a point, the main sewer is the larger of the incoming sewers. (e.g., 3.2 represents the second manhole on the main sewer from the manhole no. 3 on the trunk sewer).

Once the rough sections have been prepared, the designer should review the work for improving the spacing of manholes, the sizes and gradients of the sewers and so forth, economising on materials and excavation to the extent possible. At the same time the designer must ensure that the sewer will serve all users and that they can be actually laid according to the alignments shown in the drawing and have sufficient gradients.
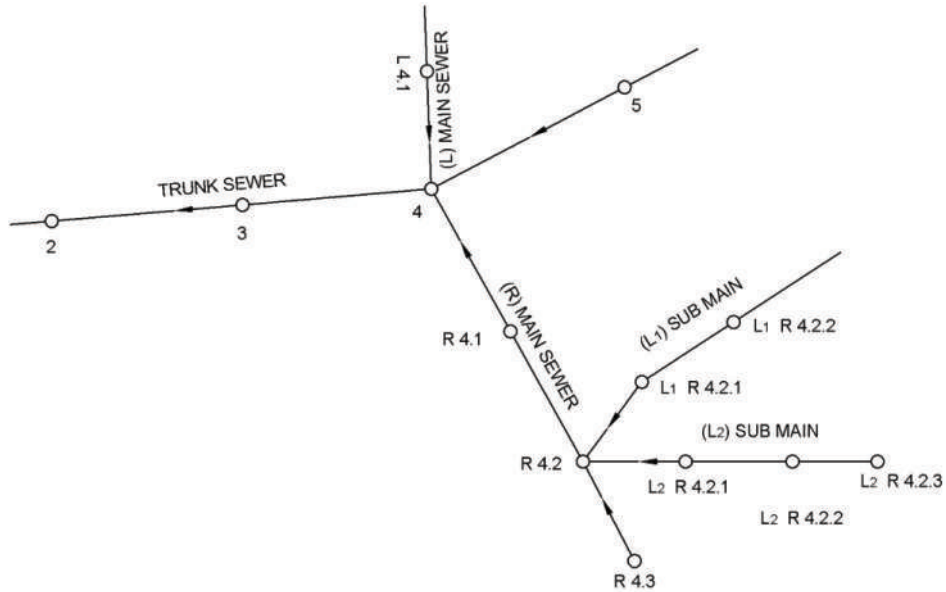
Source: CPHEEO, 1993

Figure 3.19  Nomenclature of sewers

The sewers should be shown as thick lines and the manholes as small circles in the plan. In the section, the sewer may be indicated by a line or two lines depending upon the diameters and scales adopted. Grade, size and material of pipe, ground and invert levels and extent of concrete protection should be indicated as shown in Figure 3.20.
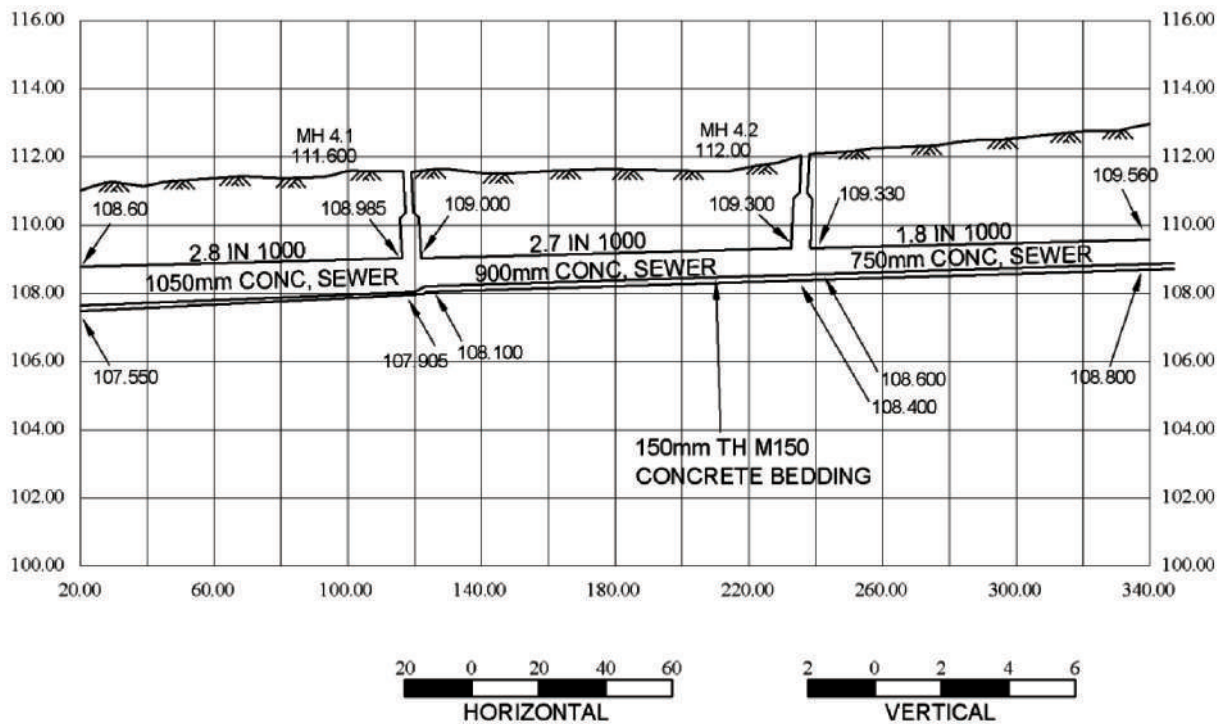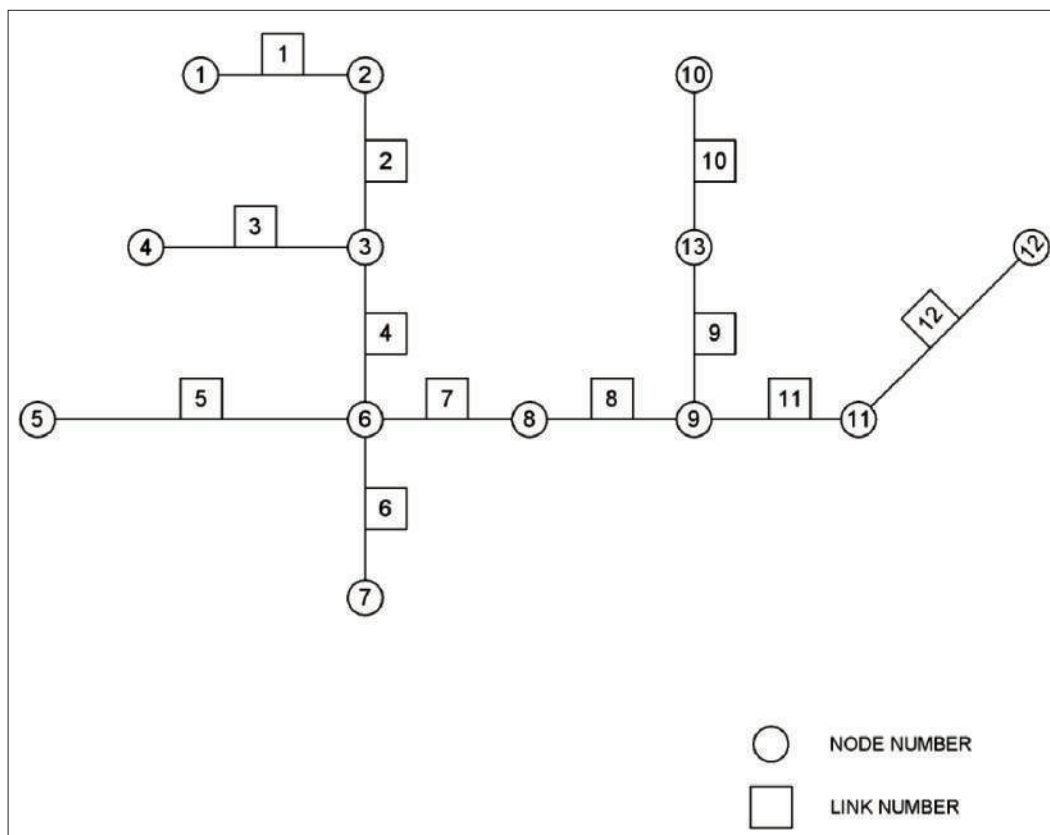


Source: CPHEEO, 1993

Figure 3.20  Typical sewer section

Standard vertical plan filing systems are now available and are very convenient for storing of plans and taking them out quickly for reference. Normally, size A0 and A1 (trimmed size 841   1,189 mm and 594   841 mm respectively) should be used along with soft copies on CD / DVD while submitting the project drawings for approval.

All documents including drawings, design calculations, measurement sheets of estimates, etc., should be in metric system. In drawings, length should be indicated either entirely in meters, corrected up to two decimals or entirely in mm (for thickness etc.). If this practice is followed, units would be obvious and in certain cases, writing of m or mm with the figure can be omitted. The flow should normally be indicated in litres per second (lps) or cubic meters per hour (m$^3$/hr) except for very large flows which may be indicated in cubic meters per second (cum/sec). For uniformity, lps for sewage flows and cum/sec for storm flows is recommended. Similarly, areas in sewer plans and design calculations may be indicated in hectares (ha). While writing figures they should be grouped into groups of three with a single space between each group and without comma. In case of a decimal number, this grouping may be on either side of the digit (e.g., 47 342.294 31).

In case of design of sewer network using computer programme, there is no restriction in the nomenclature of the sewers and manholes as required for the manual design. It is sufficient to give node numbers as well as pipe (link) numbers in any manner in the sewer network for design of the network for using computer software. The numbering of the network may be adopted as shown in Figure 3.21 and illustrated in Appendix A.3.7.



Source: CPHEEO, 1993

Figure 3.21  Example sewer network

## 3.20.7   Precautions

Design of sewer systems for rocky strata especially in hilly terrain in walled cities may have to invoke controlled blasting or chipping and chiseling both of which can cause hindrance to traffic for long periods of time and may also cause damages to heritage structures. In such situations, it is necessary to consider the shallow sewer options on both sides of the roads and if drains are already in position, construction of the additional twin of the drain and manage the collection system. The herringbone cutting for house service connections damages the roads in construction and O&M.

## PART 4  TYPES AND CONSTRUCTION OF MANHOLES

## 3.21   DEFINITION

A manhole is an opening through which a man may enter a sewer for inspection, cleaning and other maintenance and is fitted with a removable cover to withstand traffic loads in sewers. The manholes first constructed before the sewers are laid interconnecting these. The stated depths of sewers and diameter of circular manholes are in Table 3.15.

Table 3.15  Diameters of circular manholes for stated depths of sewers

| Z Values | Sulphide Condition |
|---|---|
| Z < 5,000 | Sulphide rarely generated |
| 5,000 ≤ Z ≤ 10,000 | Marginal condition for sulphide generation |
| Z > 10,000 | Sulphide generation common |

Source: CPHEEO, 1993

Note:
Where depths exceed 6 m below GL, lift stations as in Section 4.18 shall be inserted and sewage lifted to initial cover depth of 0.9 m.
In specific situations deeper depth can be justified in the DPR like outfall, etc

## 3.22   TYPES OF BRICKWORK MANHOLES

These are shown in Figure 3.22, Figure 3.23, Figure 3.24 and Figure 3.25.

## 3.23   RCC AND COMBINATION MANHOLES

In lieu of entire brickwork, RCC or RCC with brickwork combination manholes have the advantage of better quality control in raw materials and workmanship, besides easier fixing in the field with maximum speed and minimum disturbance to traffic. This is admittedly advantageous especially in case of difficulties in obtaining good bricks and the non-availability of trained masons in getting the corbelled cone portion and lapses there can lead to potential fatal accidents on public roads. There is however the issue of the concern about the concrete corrosion of the inside by sulphide gas and the soil side by sulphate in soil water. In view of this, the use of high alumina cement is advisable in manufacture itself or sulphate resistant cement with extra lining of 25 mm thickness over inner wall with high alumina cement.
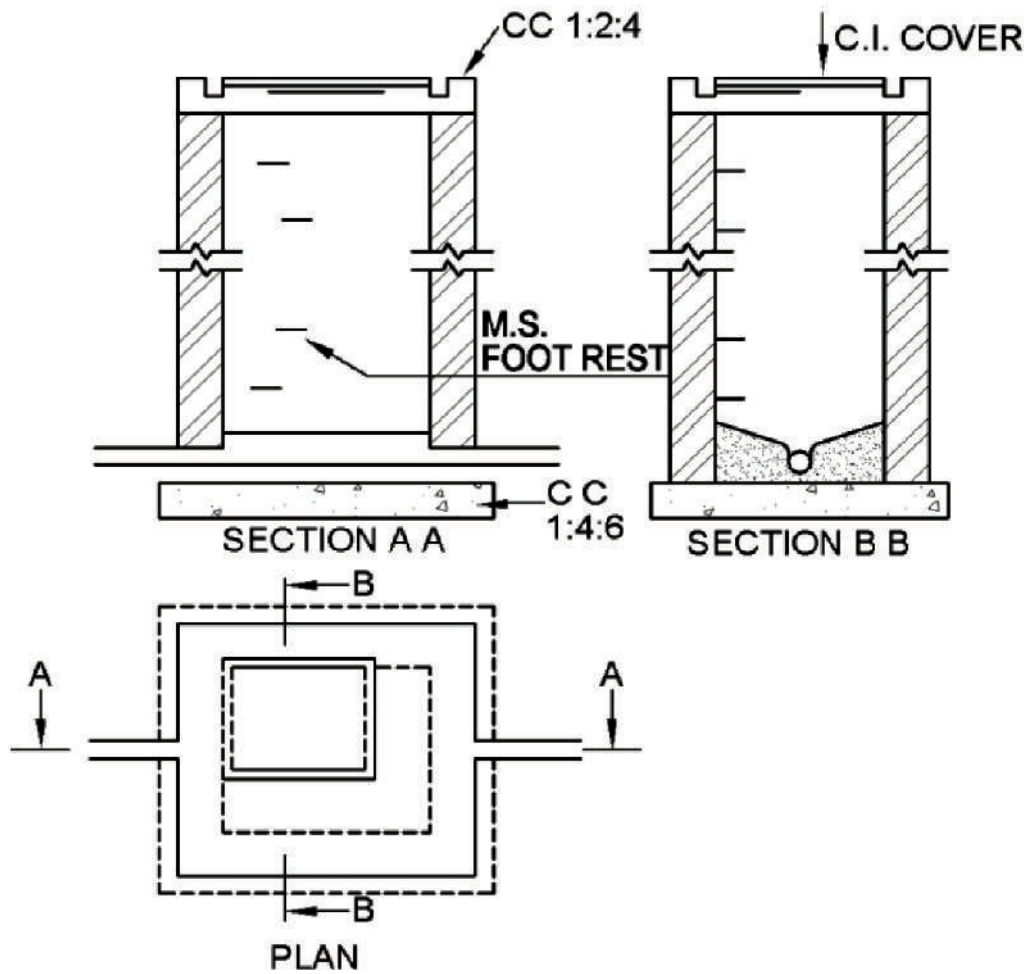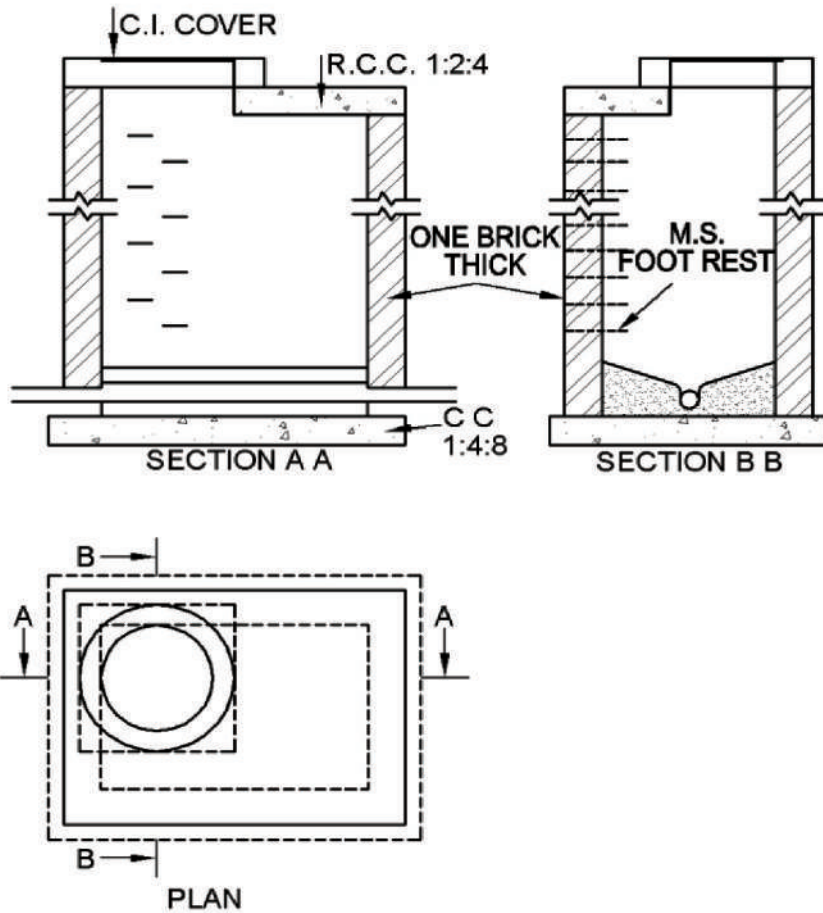
Source: CPHEEO, 1993

Figure 3.22 Rectangular manhole for 0.9 m × 0.8 m clear in plan and depth less than 0.9 m

Two types of RCC manholes can be used -

• Manholes with vertical shaft in RCC and the corbelled cone portion in brickwork

• Entire manhole in RCC and corbelled cone portion separately precast and jointed

The entries and exits of main sewers as well as house service sewers requires careful detailing because the issue of puncturing the walls for insertions of especially house service sewers later on is impossible. These shall be managed as detailed below.

• The corbelled cone portion which is eccentric with one vertical edge, shall be separately cast and its design standardized with respect to the diameter of its base.

• The vertical shaft is best pre-cast to have a better quality control of raw materials and workmanship, which is otherwise very suspect in local situations of every manhole.

• The shaft itself shall be made of rings with lap joints of the annular rim and duly jointed at site by cement mortar or elasto-polymers.
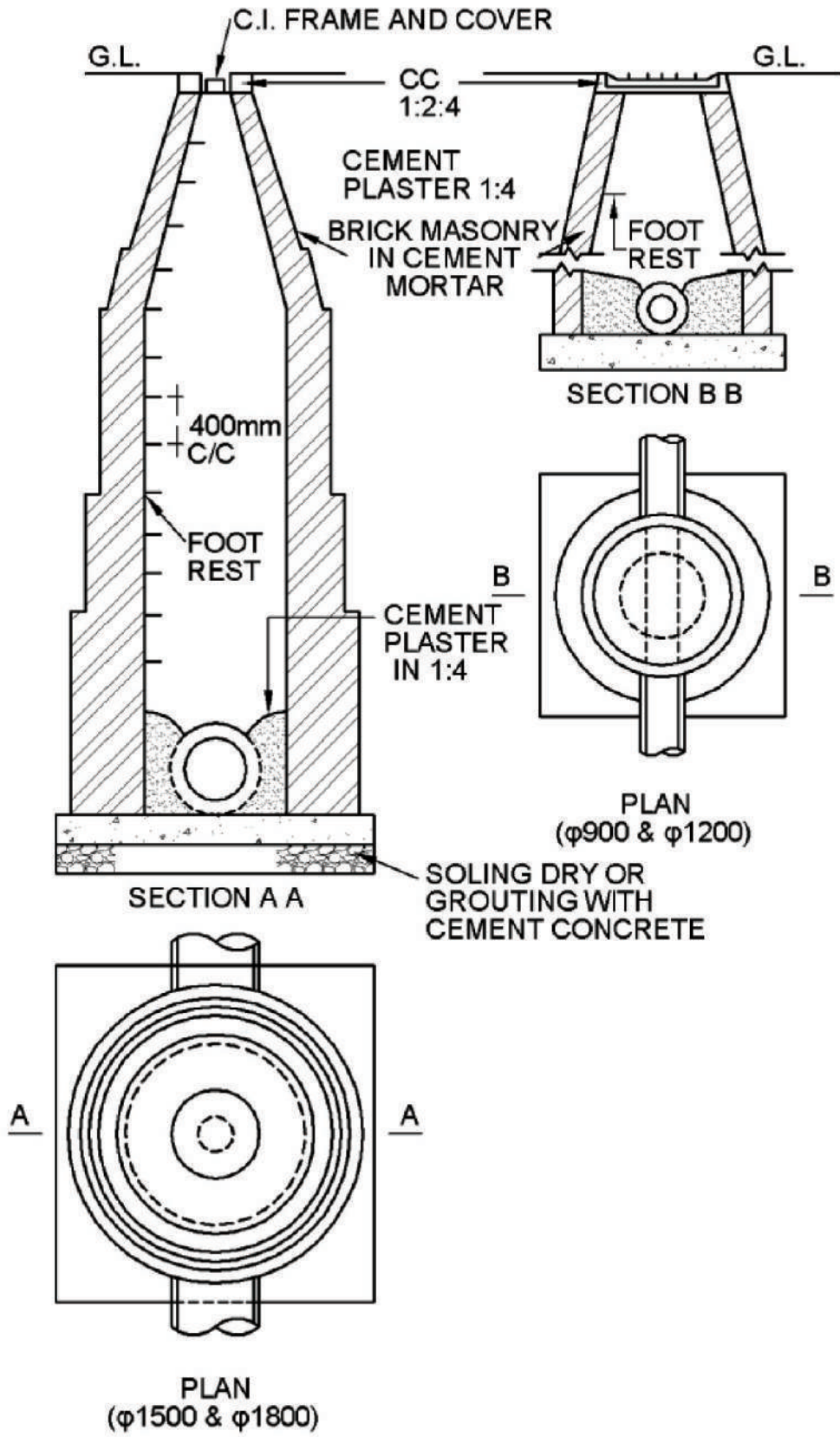
Source: CPHEEO, 1993

Figure 3.23  Rectangular manhole for 1.2 m × 0.9 m clear in plan and depth 0.9 m to 2.5 m
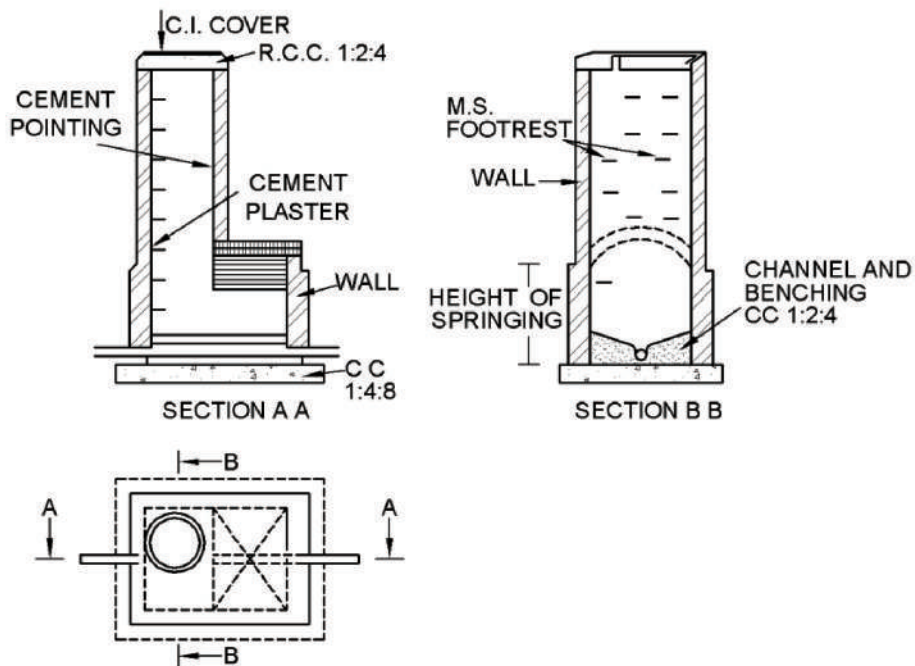


Source: CPHEEO, 1993

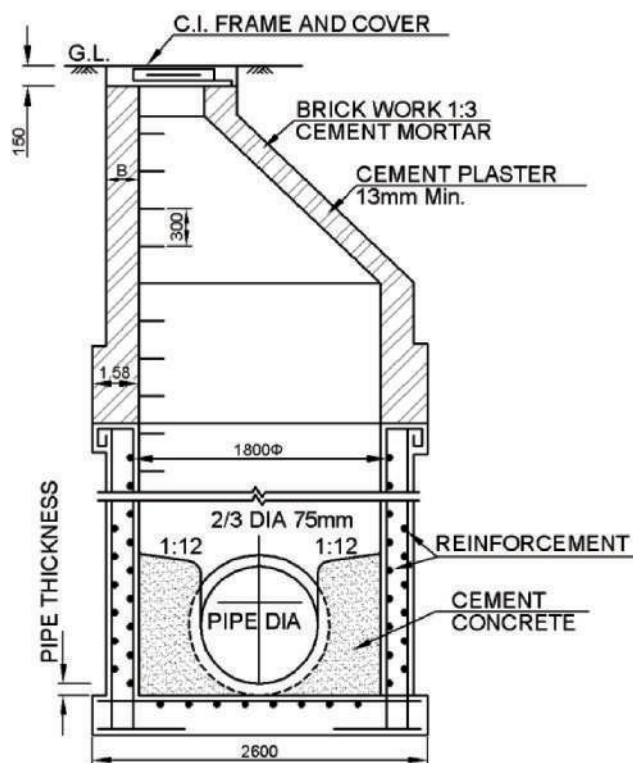Figure 3.24  Arch type manhole for 1.4 m × 0.9 m clear in plan and deeper than 2.5 m

Source:CPHEEO, 1993

Figure 3.25  Typical circular manhole of diameters and depths as in Table 3-15

- The varying heights of the manhole are obtained by choosing the bottom ring deeper than the fractional height needed there and filling up the bottom floor after placing the ring such that the invert level of the sewer is obtained thereby.

- This ring shall have a vertical inverted U cut out in casting itself to insert the sewer pipes and caulk the annular space using cement concrete with cement-based water proofing admixtures. The dimensions of the U cut out shall be standardized to match the OD of proposed sewers and a clear cover of 50 cm all round for caulking.

- The position of the vertical inverted U cut outs will normally be 180 degrees apart in plan but in cases of junction manholes and drop manholes it may be at differing angles in plan and needs to be precast suitably and shall not be chiseled out in the field.

- For insertion of the house service sewers into the manholes, it is necessary to have a precast ring section below the corbel portion, with holes at 45 degrees to the public sewer line to facilitate insertion of three house service sewers on each side of the public sewer axis. Usually the house service sewers shall be 110 mm or 160 mm UPVC 4 kg/sqcm (as detailed in sewer laying section). Accordingly, the height of the ring shall be 250 mm and 300 mm to permit filling of the annular interspaces between the sewer and the opening with cement concrete of at least 50 mm around the finished sewer.

A typical example of a combination manhole is shown in Figure 3.26, Figure 3.27 and Figure 3.28.



Source:CPHEEO, 1993

Figure 3.26  RCC and Brickwork Combination Manhole

NOTES:
1. ALL DIMENSIONS ARE IN MILLIMETRE.
2. GRADE OF CONCRETE SHALL BE M30

25mm SLOT GROUTED WITH CEMENT SLURRY WITH 8mm ROD.

TYP. PANEL DETAIL

25MM GROUT HOLE TYP. LOCATED AT 60° C/C

PLAN SHOWING GROUT HOLE LOCATION

JOINT SEALANT COMPOUND BITUMINOUS PAD.

SMOOTH PAPER FINISH

DETAIL "B"

PRECAST UNIT CONNECTION DETAIL "A"

PRECAST UNIT CONNECTION DETAIL "A"

RCPC MH COVER

GENERAL ARRANGEMENT

75 THK. PCC

150 THK. PRECAST PANEL

RING BEAM

75 THK. P.C.C 1:4:8

100 THK. BASE SLAB

DETAIL A

DETAIL B

Figure 3.27  RCC Manhole for a depth between 0.6 m to 2.3 m

Figure 3.28  RCC Manhole for a depth between 2.3 m to 5 m

## 3.24    HDPE MANHOLES

HDPE manholes with EN 13598-2: 2009 and ISO (ISO 9001: 2008) specifications are recent entrants. However, the Indian standards are yet to be brought out by BIS. These being ready made can speed up the construction as compared to brickwork manholes. However, if desired for a specific location they are to be safeguarded against the uplift pressure due to high ground water table and crushing under high traffic load etc. by suitably anchoring and, the cost of these shall not be compromised.

## 3.25    DROP MANHOLES

Difference in elevations of incoming and outgoing sewers, which would result in holding up of solids that can cause nuisance to the maintenance personnel, should be avoided. When it is necessary to drop the elevation of the sewer at a manhole, the drop should be made by means of an outside connection - in this regard, the dimensions of the related fittings govern the minimum vertical outside drop that can be made.

The designer's judgment will determine, in each case, where the difference in elevation warrants using an outside drop instead of lowering the upstream or branch sewer. The outside connection is provided for the protection of the person who may enter the manhole. Therefore, sometimes when a lateral sewer joins a deeper, sub main sewer or the use of a drop manhole will reduce the amount of excavation needed by allowing the lateral to maintain a shallow slope. The sewage drops into the lower sewer through the vertical pipe at the manhole.

Encasement of the entire outside drop in concrete or brick masonry is needed to protect it against damage during the backfilling of the trench. Maintenance may be facilitated by providing a cross instead of a tee at the top of the vertical drop, with a cast iron riser from the cross to the surface of the ground where a cast iron lamp hole frame and cover are installed. When such a drop is plugged, a ball or a chain is dropped down to break any sticks, thereby permitting the plugging material to be washed out.

When a sewer connects with another sewer, and the difference in level between the sewers of the main line and the invert level of branch line is more than 600 mm or a drop of more than 600 mm is required to be given in the same sewer line, it is uneconomical or impractical to arrange the connection within 600 mm. At that point, a drop connection shall be provided for which a manhole may be built incorporating a vertical or nearly vertical drop pipe from the higher sewer to the lower sewer.

This pipe may either be outside the shaft and encased in concrete or may be supported on brackets inside the shaft, which should be suitably enlarged.

If the drop pipe is outside the shaft, a continuation of the sewer should be built through the shaft wall to form a rodding and inspection eye. This should be provided with a half-blank flange.

If the drop pipe is inside the shaft, if should be in cast iron and it would be advantageous to provide adequate means for rodding and a cushion of 150 mm depth should also be provided.

The drop pipe should terminate at its lower end with a plain or duck-foot bend turned so as to discharge its flow at 45 degrees or less to the direction of the flow in the main sewer and the pipe, shall be cast iron, or surrounded with 150 mm of concrete. In the case of sewers that are over 450 mm in diameter the drop in level may be accomplished by one of the following methods:

a)  A Cascade

This is a steep ramp composed of steps, over which the flow is broken up and retarded. A pipe connecting the two levels is often concreted under the steps to allow small flows to pass without trickling over the steps. The cascade steps may be made of heavy duty bricks of Class I quality IS 2180, cement concrete with granolithic finish or dressed granite.

b)  A Ramp

A ramp may be formed by increasing the grade of the last length of the upper sewer to about 45 degrees or by constructing a steeply graded channel or culvert leading from the high level to the low level sewer. In order to break up the flow down the ramp and minimize the turbulence in the main sewer, the floor of a culvert ramp should be obstructed by raised transverse ribs of either brick or concrete at 1.15 m intervals and a stilling pool provided at the bottom of the ramp.

c)  By Drops in Previous Successive Manholes

Instead of providing the total drop required at the junction manhole, the same might be achieved by giving smaller drops in successive manholes preceding the junction manhole. Thus, for example, if a total drop of 2.4 m is required to be given, 0.6 m drop may be given in each of the previous three manholes and the last 0.6 m drop may be given at the junction manhole.

The diameter of the back-drop should be at least as large as that of the incoming pipe. A typical illustration of a drop manhole is shown in Figure 3-29 overleaf.

## 3.26    JUNCTION MANHOLES

A manhole should be built at every junction of two or more sewers, and the curved portions of the inverts of tributary sewers should be formed within the manhole. To achieve this with the best economy of space, the chamber may be built of a shape other than rectangular. The soffit of the smaller sewer at a junction should not be lower than that of the larger sewer, in order to avoid the surcharging of the former when the latter is running full, and the hydraulic design usually assumes such a condition. The gradient of the smaller sewer may be steepened from the previous manhole sufficiently to reduce the difference of invert level at the point of junction to a convenient amount.

## 3.27    SIDE ENTRANCE MANHOLES

In large sewers or where it is difficult to obtain direct vertical access to the sewer from ground level, owing to existing services, gas, water, etc., the access shaft should be constructed in the nearest convenient position off the line of sewer, and connected to the manhole chamber by a lateral passage.
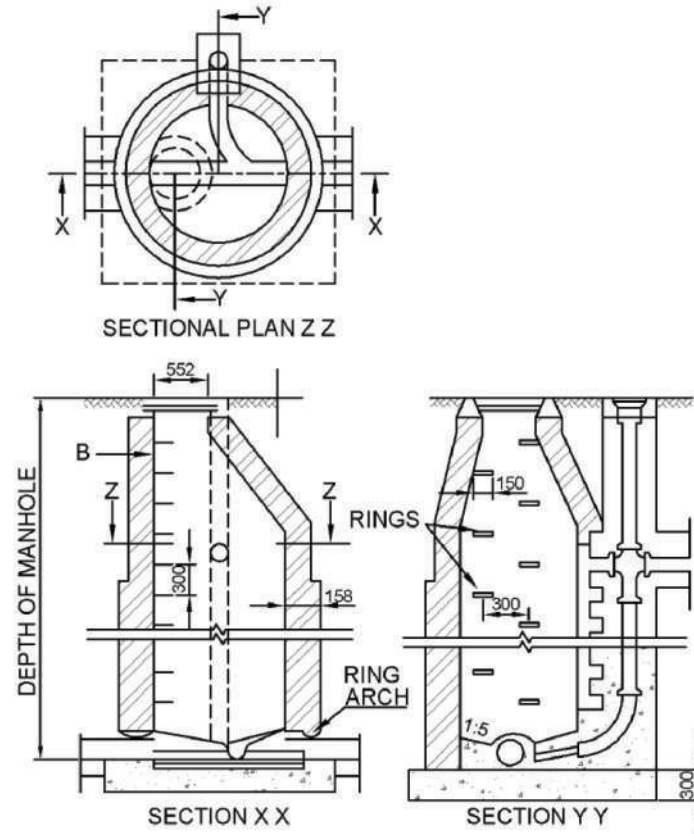
Source: CPHEEO, 1993

Figure 3.29  Illustrative arrangement of drop manhole

In the tunnelled sewers the shaft and lateral access heading may be used as a working shaft, the tunnel being broken out from the end of the heading, or alternatively the shaft and heading may be constructed after the main tunnel is complete, provision having been made for breaking in from the access heading to build the chamber.

The floor of the side-entrance passage, which should fall at about 1 in 30 towards the sewer, should enter the chamber not lower than the soffit level of the sewer. In large sewers where the floor of the side entrance passage is above the soffit, either steps or a ladder (which should be protected either by a removable handrail or by safety chains) should be provided to reach the benching.

## 3.28    SCRAPPER (SERVICE) TYPE MANHOLE

It has been proposed in the earlier 1993 edition of the manual that the scraper manholes shall be used at specified intervals for desilting the sewer systems.

In the interim period, advancements in mechanized sewer cleaning, like jet rodding and vacuum suction machines have occurred and are being used. In order to avoid man entry into sewer manholes these scraper manholes shall be discontinued forthwith. Instead, the numbers of these mechanized sewer-cleaning equipment as recommended in Part B of the O&M manual shall be included in the project in the TOR stage itself and procured.

## 3.29    FLUSHING MANHOLES

Where it is not possible to obtain self-cleansing velocities due to flatness of the gradient especially at the top ends of branch sewers, which receive very little flow, it is essential that some form of flushing device be incorporated in the system. This can be done by making grooves at intervals of 45 m to 50 m in the main drains in which wooden planks are inserted and water allowed to head up and which will rush on with great velocity when the planks are removed. Alternatively, an elevated tank is built and filled with treated sewage from which connections are made through pipes and flushing hydrants to rush water to the sewers. The relevant Indian Standard is IS 4111 Part 2. Flushing can also be very conveniently accomplished by the use of a fire hydrant or tanker and hose.

Where flushing manholes are provided, they are located generally at the head of a sewer. Sufficient velocity shall be imparted in the sewer to wash away the deposited solids. The flush is usually effective up to a certain distance after which the imparted velocity gets dissipated.

The automatic systems, which are operated by mechanical units often get corroded by the sewer gases and do not generally function satisfactorily and hence, are not recommended.In case of hard choking in sewers, care should be exercised to ensure that there is no possibility of backflow of sewage into the ground and entering defective water supply mains.

Approximate quantities of water needed for flushing are given in Table 3.16

Table 3.16  Quantity of water needed for flushing

| | Quantity of water (Litres) | | |
|---|---|---|---|
| Slope | 200 mm dia. | 250 mm dia. | 300 mm dia. |
| 1 in 200 | 2,300 | 2,500 | 3,000 |
| 1 in 130 | 1,500 | 1,800 | 2,300 |
| 1 in 100 | 1,300 | 1,500 | 2,000 |
| 1 in 50 | 500 | 800 | 1,000 |
| 1 in 33 | 400 | 500 | 700 |

Source:CPHEEO, 1993

A more practical and relatively safer method is to deploy the modern jet rodding machines at head manholes and use the treated sewage from the STP, but then, the cost of the machine is involved. A simpler option will be to use the possibility of positioning a butterfly valve at the head sewer mouth in the manhole and is kept open by an extended handle, which can be operated from ground level when the manhole cover is opened. After opening the manhole cover, the valve is closed by a quarter turn and the manhole is filled with treated sewage brought by a tanker from the STP. After filling, the valve is opened to enable flushing. Usually a sewer tanker can hold about 6000 litres and is adequate to flush at least two manholes per trip.

## 3.30    DIFFERENT DIAMETERS OF SEWERS IN THE SAME MANHOLE

Manholes should be built to cause minimum head loss and interference with the hydraulics of the sewer line. One way to maintain a relatively smooth flow transition through the manhole, when a small sewer joins one of a larger diameter, is to match the pipe crown elevations at the manhole.

## 3.31    TERMINAL CLEANOUT STRUCTURE

Terminal cleanouts are sometimes used at the ends of branch or lateral sewers. Their purpose is to provide a means for inserting cleaning tools for flushing or for inserting an inspection light into the sewer. In fact, a terminal cleanout amounts to an upturned pipe coming to the surface of the ground. The turn is made with bends so that flexible cleaning rods can be passed through them. The diameter of a bend should be the same as that of the sewer. The cleanout is capped with a cast iron frame and cover. Care should be taken to maintain proper alignment of the pipe while encasing it with concrete. The frame and cover of the terminal cleanout structure should be made of grey cast iron. Tees were often used, instead of pipe bends, in older engineering practice, and the structures were called "lamp holes". Modern sewer cleaning equipment cannot be passed from the surface through such structures, so their use is to be discontinued forthwith. Terminal cleanouts are limited in usefulness and should never be used as a substitute for a manhole. They are permitted under some state regulations only at the ends of branch sewers, which may never be extended and must lie within 50 m of a manhole.

## 3.32    CONSTRUCTION OF BRICKWORK MANHOLES

a) If the sewer is constructed in a tunnel, the manhole should be located at the access or working shafts and the manhole chamber may be constructed of a size to suit the working shaft or vice-versa. The width/diameter of the manhole should not be less than internal diameter of the sewer + 150 mm benching on both sides (150 mm + 150 mm).

b) The opening for entry into the manhole (without cover) should be of such minimum dimensions as to allow a work with the cleaning equipment to get access into the interior of the manhole without difficulty. A circular opening is generally preferred. A minimum clear opening of 60 cm is recommended. Suitable steps, usually of malleable cast iron shall be provided for entry.

c) Access shaft shall be circular in shape and shall have a minimum internal diameter of 750 mm; where the depth of the shaft exceeds 3 m suitable dimensions shall be provided to facilitate cleaning and maintenance. Access shaft where built of brickwork, should be corbelled on three sides to reduce it to the size of the opening in the cover frame, and to provide easy access on the fourth side to step irons or ladder. In determining the sizes, the dimensions of the maintenance equipment likely to be used in the sewers, shall be kept in view.

d) The manhole base slab shall be 150 mm for manholes up to 1 m depth, 200 mm for manholes from 1 to 2 m depth and 300 mm for greater depths. In all cases, the thickness shall be counter checked for uplift conditions based on maximum ground water elevations at the site on the soil side by considering empty manhole conditions.

e) Where subsoil water condition exists, a rich mix may be used and it shall further be waterproofed with addition of approved water proofing compound in a quantity as per manufacturer's specifications.

f) The brickwork manholes shall be first constructed to the required invert and with circular openings to facilitate the laying of sewer pipes later on. These manholes facilitate the judgement in the field when trenches are dug up and sewer pipes are to be laid to give the levels using a levelling instrument or with boning rods.

g) All brickwork shaft shall be in English bond and the jointing faces being well buttered with cement mortar before laying, so as to ensure a full joint and brickwork shall be in accordance with IS 2212 code of practice for brickwork. The cement mortar used shall not be weaker than 1:3 and in accordance with IS 2250 code of practice for preparation and use of masonry mortars and its revisions.

h) The thickness of walls shall be typically one brick (23 cm) for up to l.5 m deep manholes and one and a half brick (35 mm) for depths greater than 1.5 m. The actual thickness in any case shall be verified on the basis of engineering design in difficult soil conditions.

i) The jointing of brickwork and plastering on both sides (20 mm) shall be in a mix of cement mortar 1:3. Admixtures for water proofing if desired shall be cement based.

j) Salt glazed or 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 sewer. These can also be finished as semi-circular channels with cement mortar 1:2 and of diameter equal to that of the sewer. Above the horizontal diameter, the sides shall be extended vertically to the same level as the crown of the outgoing pipe and the top edge shall be sloped up at 1:10 towards the wall and suitably rounded off. The branch channels shall also be similar.

k) Bricks on edge shall be cut to a proper form and laid around the upper half of all the pipes entering or leaving the manhole, to form an arch.

l) All around the pipe there shall be a joint of cement mortar 12 mm thick between the pipe and the bricks. The ends of the pipes shall be built in and neatly finished off with cement mortar.

m) The entire height of the manhole shall be tested for water-tightness by closing both the incoming and outgoing ends of the sewer and filling the manhole with water. A drop in water level not more than 50 mm per 24 hours shall be permitted.

n) 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.

o) The top of the manhole shall be flush with the finished road level as per IS 4111 Part I.

## 3.33  CONSTRUCTION OF RCC MANHOLES

The idea of RCC manholes is essentially to quicken the work of construction in the roads by adopting precast sections assembled at site.

Part A: Engineering

CHAPTER 3: DESIGN AND
CONSTRUCTION OF SEWERS

Thus, the issues related to their construction are more of design itself and quality control in casting. The provisions of IS 456 and IS 3370 Parts I, II and IV shall inter alia apply to the design. The entire structure shall at all times be designed to the condition where the ground water is at ground level itself and the inside is empty and there is no superimposed load on the manhole to counter the uplift force and not considering the skin friction of the manhole sidewall with the soil.

## 3.34    COVERS AND FRAMES

The size of manhole covers should be such that there should be clear opening of not less than 560 mm diameter for manholes exceeding 0.9 m depth. When cast iron manhole covers and frames are used they shall conform to IS 1726. The frames of manhole shall be firmly embedded to correct alignment and level in plain concrete on the top of masonry. The precast frame and cover can also be of steel fibre reinforced concrete (SFRC) conforming to IS 12592 and shall be of approved make. The frame and cover shall be of LD/ MD/ HD/ EHD grade, size and thickness as mentioned in the description of the item. The standard for DI manhole covers is EN 123.

## 3.35    RUNGS

As per the US Department of Environmental Conservation, Model Sewer Use Law, Section 504, Manholes and Manhole Installation clause 6, "No steps or ladder rungs shall be installed in the inside or outside manhole walls at any time" (http://www.dec.ny.gov/chemical/8729.html). This implies the total ban on man entry into manholes (leave alone the nomenclature) and in turn underscores the fully mechanized methods of attending to sewer blocks. Though it is the ideal condition, the relatively lesser per capita water supply rates and the absence of strict enforcement of grinders below faucets in kitchens etc. and handicapped financial positions of local bodies defy the adoption of such an ideal situation in our country for some more time and may be adopted in stages starting from mega cities. Till such time, the rungs shall continue to be used.

Where the depth of the manhole exceeds 90 cm below the surface of the ground, rungs shall be built into the brickwork. The vertical distance between the two consecutive rungs shall not be more than 30 cm and the centre-to-centre horizontal distance between alternate rungs shall not exceed 38 cm. The rungs shall have a width of 15 cm.

## 3.36    PILE SUPPORTS IN LOOSE SOILS

Where the soil is weak, RCC piles shall be driven to hard stratum and the pile cap made of the same size of the PCC of the manhole and after pouring the RCC for the pile and capping slab, immediately the RCC for foundation of the manhole shall also be poured and integrally cured. Tremie pipe shall be used along with bentonite lining as the case may be. The typical diagram for the RCC pile support is in Figure 3-30 overleaf.

## 3.37    MANHOLE REHABILITATION

While preparing DPR for augmentation of sewerage in already existing sewered habitations, it is necessary to look into the needs of rehabilitation of the old system and include the appropriate financial provisions.

3 - 59
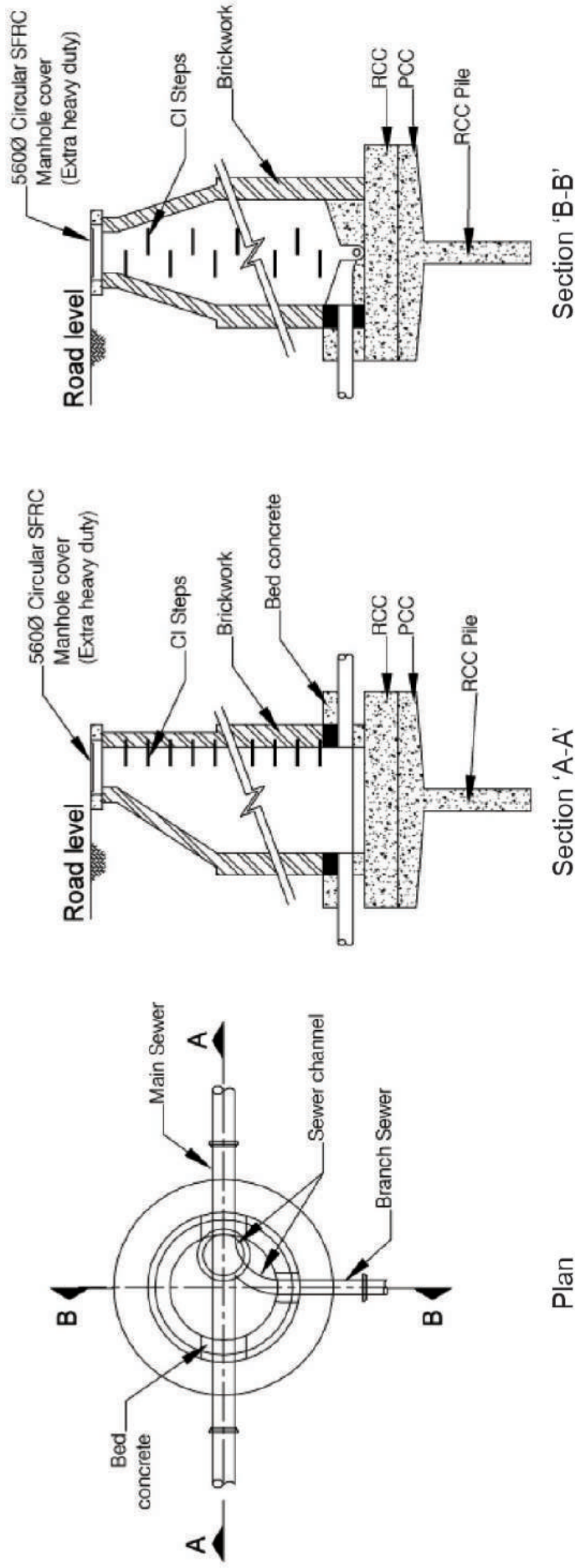
Pile will be of RCC. Driving through soil water will need bentonite casing and pouring using tremmie pipe. Sulphate resistant cement is best used here. Pile should be driven to hard strata irrespective of depth. RCC and PCC to be poured intergrally and concrted using sulphate resistant cement

Source: CPHEEO, 1993

Figure 3.30  Illustrative arrangement of manholes in loose soils or slushy soils or quick sand

With sewer systems in our country dating back to as old as 75 years and all manholes being of brickwork, there is a need to look into the manhole rehabilitation contingencies. The following approach is recommended.

a) Institute an ultrasonic survey of the structural integrity of all manholes known to be more than 30 years, the accepted life cycle of civil works and maintain an annual repeat record, which will indicate the manholes requiring immediate attention.

b) Isolate the manhole from service by plugging the sewers with inflatable special balloons and transfer pumping from upstream manhole to downstream manhole using submersible pump sets in the upstream manhole; prepare the surface by cleaning it and removing loose particles.

c) Adopt the lining of the insides by the commercially available fast-curing elastomeric/other material that can be directly applied to existing concrete or brickwork using spray techniques whereby a homogeneous, non-porous and monolithic lining is formed. This can provide effective surface protection against wear, corrosion and water infiltration.

d) This will mark a new era in sustaining the infrastructures created in sewer manholes and forestall major issues when the rehabilitation needs arise suddenly.

e) Recent technologies provide for spray lining of the manholes without man entry. A polymeric/elastomer material like polyurea is obtained as a powder and a solution is made at site and is pumped through a vertical guide pipe in the central axis with a spray nozzle at its base and rotating full 360 degrees in plan. The thickness of lining is controlled by the rate of solution pumped, the revolutions per minute and the rate of rise of the guide pipe. There is no need to block the entry and exit sewers, as the spray entering these under pressure will line these pipes also to a certain length as well. An illustration is shown in Figure 3.31.
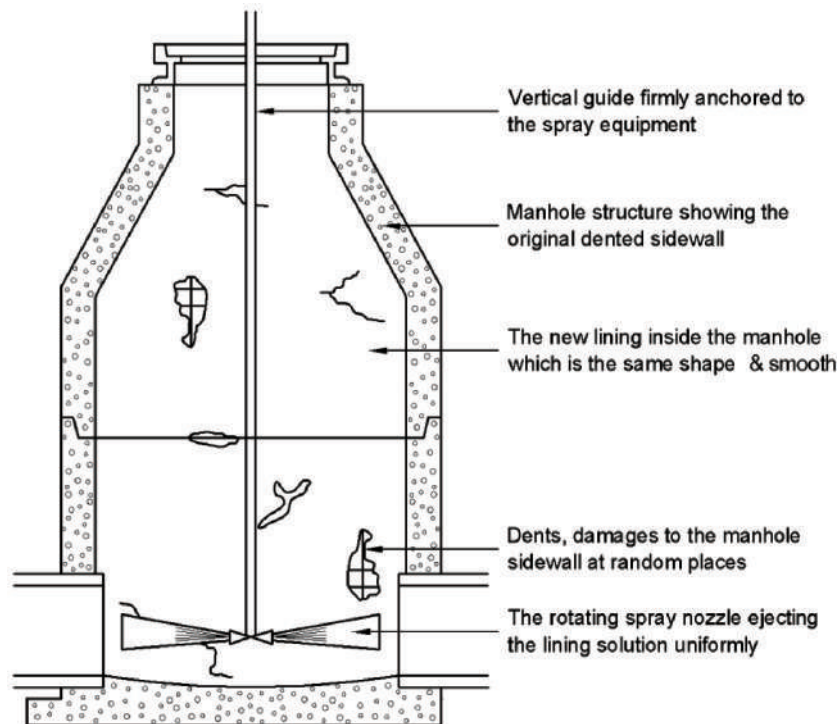


Figure 3.31  Depiction on in-situ sewer manhole lining (Spray equipment not shown)

**PART - 5 LAYING, JOINTING AND CONSTRUCTION OF SEWERS**

### 3.38    STAGING OF SEWERAGE WORKS

Due to enormous scope of sewerage work, it is desirable to set up priorities for taking up the works of different component for execution. It is generally noticed that most of sewerage schemes are not completed for want of funds, land, as well as due to public litigation and execution of work in improper sequence. The partially executed schemes could not be made functional. Therefore, the priorities of works shall be followed during execution in sequence as shown below.

(1)   Sewage treatment plants

(2)   Trunk mains

(3)   Sewage pumping stations (if required)

(4)   Main sewers

(5)   Sub main sewers

(6)   Sewers (Laterals)

The works at Sr. No. 1 to 3 mentioned above can be taken simultaneously. However, only after completion of all works from Sr. 1 to 6 mentioned above, the property connections shall be given. In case, part of main sewer or sub main sewer is not laid for want of land acquisition issues or any public litigation, the work of sewer lines joining that particular sub main/main sewer shall be postponed. Following such priorities, the executed works could be put into use, thus the expenditure made on structures shall not be proved unfruitful.

### 3.39    SEWER CONSTRUCTION

In sewer construction work, two operations are of special importance, namely, excavation of trenches, and laying of sewer pipes in trenches and tunnels. Most of the trench work involves open cut excavation; and in urban areas, it includes:

1) Removing pavement

2) Removal of the material from the ground, and its separation, its classification where necessary, and its final disposal

3) Sheeting and bracing the sides of the trench

4) Removal of water (if any) from the trench

5) Protection of other structures, both underground and on the surface, whose foundations may be affected

6) Backfilling, and

7) Replacement of the pavement

The most common type of sewer construction practice involves the use of open trenches and prefabricated pipes. However, larger sewer systems, and unusual situations may require tunnelling, jacking of pipes through the soil, or cast-in-situ concrete sewers.

On all excavation work, safety precautions for the protection of life and property are essential; and measures to avoid inconveniences to the public are desirable. Such measures and precautions include the erection and maintenance of signs (to forewarn public), barricades, bridges and detours; placing and maintenance of lights both for illumination and also as danger signals; provision of watchmen to exclude unauthorised persons, particularly children, from trespassing on the work; and such other precautions as local conditions may dictate.

 (i) Each pipe section should be uncracked.

(ii) Proper placement (i.e., bedding) has to be there for each pipe section that is laid.

(iii) There should be proper joining of pipe sections.

(iv) There should be proper alignment (direction and longitudinal slope) of the line.

(v) Pipes should be covered properly with clean fill material (backfilled).

The structural design of a sewer is based on the relationship: the supporting strength of the sewer as installed divided by a suitable factor of safety which must equal or exceed the load imposed on it by the weight of earth and any superimposed loads.

The essential steps in the design and construction of buried sewers or conduits to provide safe installations are therefore:

i) Determination of the maximum load that will be applied to the pipe based on the trench and backfill conditions and the live loads to be encountered

ii) Computation of the safe load carrying capacity of the pipe when installed and bedded in the manner to be specified using a suitable factor of safety and making certain the design supporting strength thus obtained is greater than the maximum load to be applied

iii) Specifying the maximum trench widths, the type of pipe bedding and the manner in which the backfill is to be made in accordance with the conditions used in the design

iv) Checking each pipe for structural defects before installation and making sure that only sound pipes are installed and

v) Ensuring by adequate inspection and engineering supervision that all trench widths, sub grade work, bedding, pipe laying and backfilling are in accordance with design assumptions as set forth in the project specifications.

Proper design and adequate specifications alone are not enough to ensure protection from dangerous or destructive overloading of pipe. Effective value of these depends on the degree to which the design assumptions are realized in actual construction. For this reason, thorough and competent inspection is necessary to ensure that the installation conforms to the design requirements and specifications.

## 3.40    TYPE OF LOADS

In a buried sewer, stresses are induced by external loads and by internal pressure in case of a pressure main. The stress due to external loads is of utmost importance and may be the only one considered in the design. Besides, if the sewer is exposed to sunlight, temperature stresses induced may be considerable and these will have to be taken into consideration particularly in case of metallic pipes. The external loads are of two categories viz., load due to backfill material known as backfill load and superimposed load, which again is of two types viz. concentrated load and distributed load. Moving loads may be considered as equivalent to uniformly distributed load. Sewer lines are mostly constructed of stoneware, concrete or cast iron, which are considered as rigid pipes (while steel pipes, if used, are not considered as rigid pipes). The flexibility of the pipe affects the load imposed on the pipe and the stresses induced in it.

## 3.41    LOADS ON CONDUIT DUE TO BACKFILL

Methods for determining the vertical load on buried conduits due to gravity earth forces in all commonly encountered conditions, as developed by A. Marston, are generally accepted as the most suitable and reliable for computation. Theoretically stated, the load on a buried conduit is equal to the weight of the prism of earth directly over the conduit, called the interior prism of earth plus or minus the frictional shearing forces transferred to the prism by the adjacent prism of earth.

The considerations are:

a) The calculated load due to the backfill is the load that will develop when ultimate settlement has taken place.

b) The magnitude of the lateral pressure causing the shearing force is computed by Rankine's theory.

c) There is negligible cohesion except for tunnel conditions.

The general form of Marston's formula is

$$W = C\,w B^2 \tag{3.23}$$

where,

$W$ : Vertical load in kgs per metre length acting on the conduit due to gravity earth loads

$w$ : Unit weight of earth, kg/m$^3$

$B$ : Width of trench or conduit in meters depending upon the type of installation conditions, m

$C$ : Dimensionless co-efficient that measures the effect of

   a) Ratio of height of fill to width of trench or conduit

   b) Shearing forces between interior and adjacent earth prisms and

   c) Direction and amount of relative settlement between interior and adjacent earth prisms for embankment conditions.

**3.42    TYPES OF INSTALLATION OR CONSTRUCTION CONDITIONS**

The accepted types of installation or construction conditions are shown in Figure 3.32. There are three classifications for the construction conditions:



Source: CPHEEO, 1993

Figure 3.32  Classification of construction conditions

1)  Embankment condition

2)  Trench condition and

3)  Tunnel condition

Embankment condition prevails when the conduit is covered with fill above the original ground surface or when a trench in undisturbed ground is so wide that trench wall friction does not affect the load on the pipe. The embankment condition is further classified depending upon the position of the top of conduit in relation to the original ground surface as

i)     Positive projecting condition

ii)    Zero projecting condition

iii)   Negative projecting condition and

iv)   Imperfect trench condition

Trench condition exists when the pipe or conduit is installed in a relatively narrow trench (not wider than twice the external diameter of the pipe) cut in undisturbed soil and then covered with earth backfill up to the original ground surface. Tunnel condition exists when the sewer is placed by means of jacking or tunnelling.

## 3.43    LOADS FOR DIFFERENT CONDITIONS

### 3.43.1    Embankment or Projecting Conduit Condition

a)  Positive Projecting Conduit

A conduit is said to be laid as a positive projecting conduit when the top of the conduit is projecting above the natural ground into the overlying embankment (Figure 3-33).
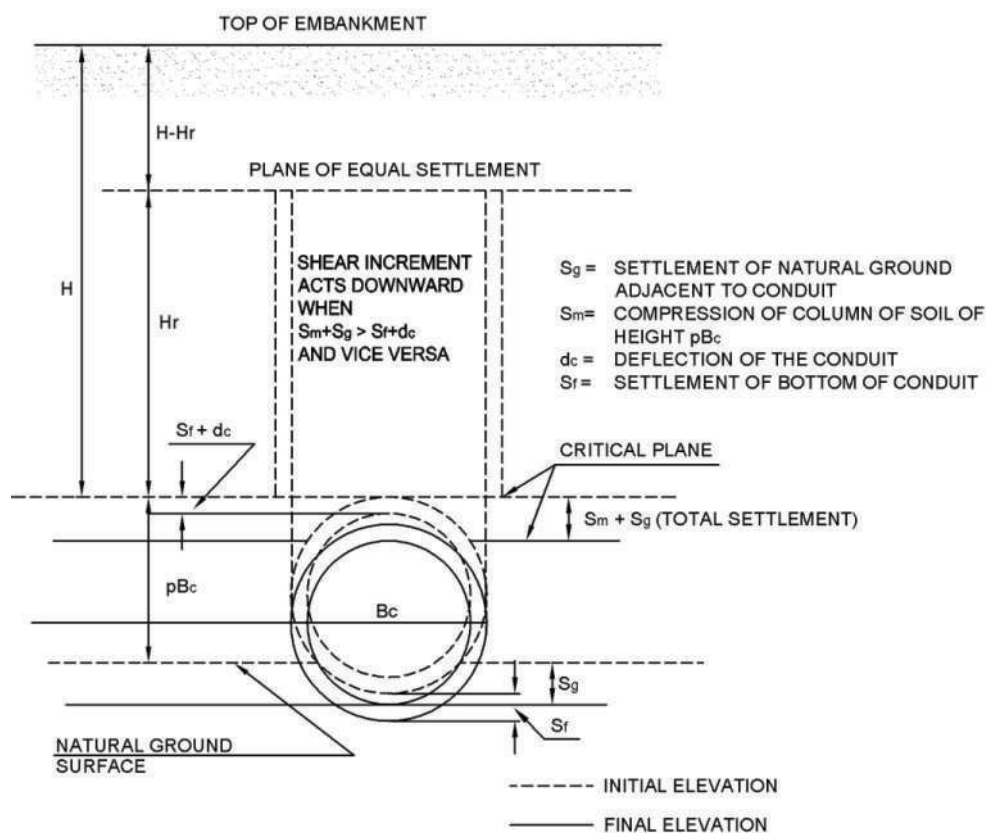


Source: CPHEEO, 1993

Figure 3.33  Settlements that influence loads on positive projecting conduits

i)  Load Producing Forces

The load on the positive projecting conduit is equal to the weight of the prism of soil directly above the structure plus or minus vertical shearing forces, which act in a vertical plane extending upward into the embankment from the sides of the conduit. These vertical shearing forces ordinarily do not extend to the top of the embankment but terminate in a horizontal plane at some elevation above the top of the conduit known as the plane of equal settlement as shown in Figure 3.33, which also shows the elements of settlement ratio.

$$Settlement\ ratio\ r_{SD} = \frac{Settlement\ of\ critical\ plane - settlement\ of\ top\ of\ conduit}{Compression\ of\ height\ of\ column\ H\ of\ embankment}$$  (3.24)

$$= \frac{(S_m + S_g) - (S_f + d_c)}{S_m}$$

where,

H : Height of top of conduit above adjacent natural ground surface (initial) or the bottom of a wide trench

p. Bc: where p is the projection ratio and Bc is the outside width of conduit

Sm : Compression column of height H of embankment

Sg : Settlement of natural ground adjacent to the conduit

Sf : Settlement of the bottom of conduit and

dc : Deflection of conduit or shortening of its vertical height under load.

When $(S_m + S_g) > (S_f + d_c)$, $r_{SD}$ is positive, i.e., the shearing forces act downwards. Therefore, the load on conduit is equal to weight of critical prism plus shear force. When $(S_m + S_g) < (S_f + d_c)$, $r_{SD}$ is negative and the shear force acts in the upward direction. The settlement ratio $r_{SD}$ therefore, indicates the direction and magnitude of the relative settlement of the prism of earth directly above and adjoining the conduit. The product of $r_{SD}$ and p gives the relative height of plane of equal settlement and hence of the magnitude of the shear component of the load. When $r_{SD} \times p = 0$, the plane of equal settlement coincides with the critical plane and there are no shearing forces and the load is equal to the weight of the central prism. It is not practicable to predetermine the rsd value. The recommended design values based on actual experience are given in Table 3.17.

Table 3.17  Recommended design values of settlement ratios

|   | Type of Conduit | Type of Soil | Settlement Ratio |
|---|---|---|---|
| 1 | Rigid | Rock or unyielding foundation | + 1.0 |
| 2 | Rigid | Ordinary foundation | + 0.5 to + 0.8 |
| 3 | Rigid | Yielding foundation | 0 to + 0.5 |
| 4 | Rigid | Negative projecting installation | 0.3 to 0.5 |
| 5 | Flexible | Poorly compacted side fill | 0.4 to 0 |
| 6 | Flexible | Well compacted side fill | 0 |

Source:CPHEEO, 1993

ii) Computation of Loads

Marston's formula for positive projecting conduits (both rigid and flexible) is mentioned overleaf.

$$W_c = C_c \, w \, B_c^{\,2}$$ (3.25)

where,

$W_c$ :    Load on conduit in kg/m

$w$ :    Unit weight of backfill material in kg/m$^3$

$B_c$ :    Outside width of conduit in m

$C_c$ :    Load coefficient, which is a function of the product of the projection ratio and the settlement ratio and ratio of the height of fill above the top of the conduit to the outside width of the conduit (H/B). It is also influenced by the coefficient of internal friction of the backfill material and the Rankine's ratio of lateral pressure to vertical pressure Ku. Suggested values for Ku for positive and negative settlement ratios are 0.19 and 0. 13, respectively.

The value of $C_c$ can be obtained from Figure 3.34.
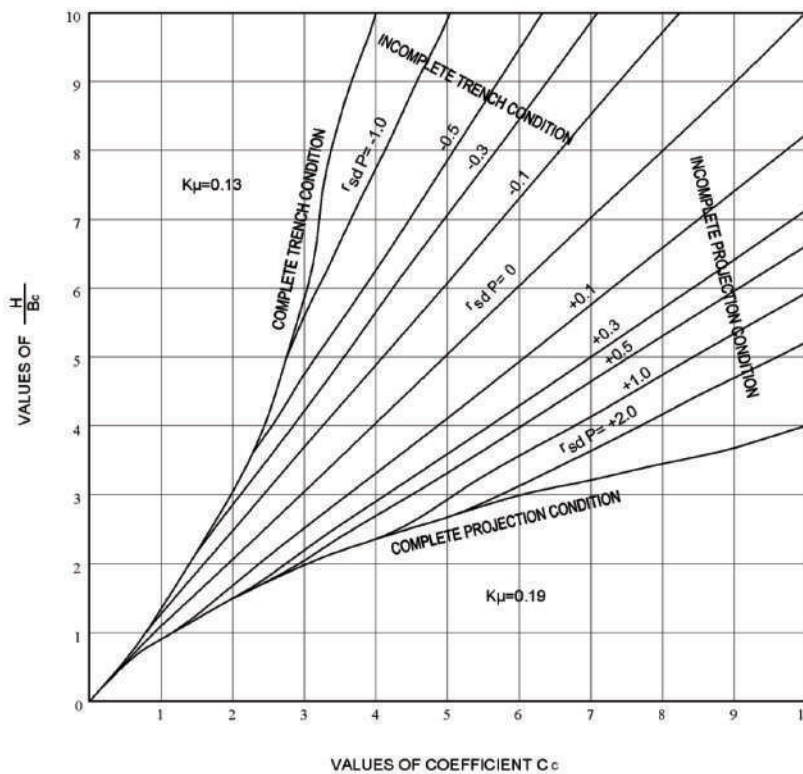
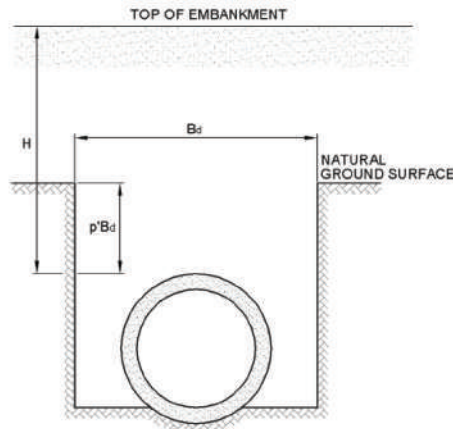

Source:  CPHEEO, 1993

Figure 3.34  Diagram for coefficient Cc for positive projecting conduits

b)  Negative Projecting Conduit

A conduit is said to be laid in a negative projecting condition when it is laid in a trench, which is narrow with respect to the size of pipe and shallow with respect to depth of cover. Moreover, the native material of the trench is of sufficient strength that the trench shape can be maintained dependably during the placing of the embankment, the top of the conduit being below the natural ground surface

and the trench refilled with loose material and the embankment constructed above (Figure 3.35). The prism of soil above the conduit, being loose and greater in depth compared to the adjoining embankment, will settle more than the prism over the adjoining areas thus generating upward shear forces which relieve or reduce the load on the conduit.



Source: CPHEEO, 1993

Figure 3.35  Negative projecting conduit

i) Computation of Loads

Marston's formula for negative projecting conduits is given by

$$W_c = C_n \, w \, B_d^{\,2}$$ (3.26)

where,

$W_c$ :     Load on the conduit in kg/m

$Bd$ :     Width of trench in m

$w$ :     Unit weight of soil in kg/m³

$C_n$ :     Load coefficient, which is a function of the ratio (H/Bd) of the height of fill and the width of trench equal to the projection ratio p (Vertical distance from the firm ground surface down to the top of the conduit/width of the trench) and the settlement ratio rsd given by the expression,

$$r_{sd} = \frac{settlement \ of \ natural \ ground - settlement \ of \ critical \ plane}{compression \ of \ the \ backfill \ within \ the \ height \ p \, B_d}$$ (3.27)

$$= \frac{S_g - \left(S_d + S_f + d_c\right)}{S_d}$$

Values of $C_n$, for various values of H/$B_d$, $r_{sd}$, and p are given in Figure 3.36 overleaf.

Exact determination of the settlement ratio is very difficult. Recommended value of $r_{sd}$ is 0.3 for design purposes. Elements of settlement ratios are shown in Figure 3.37 overleaf.
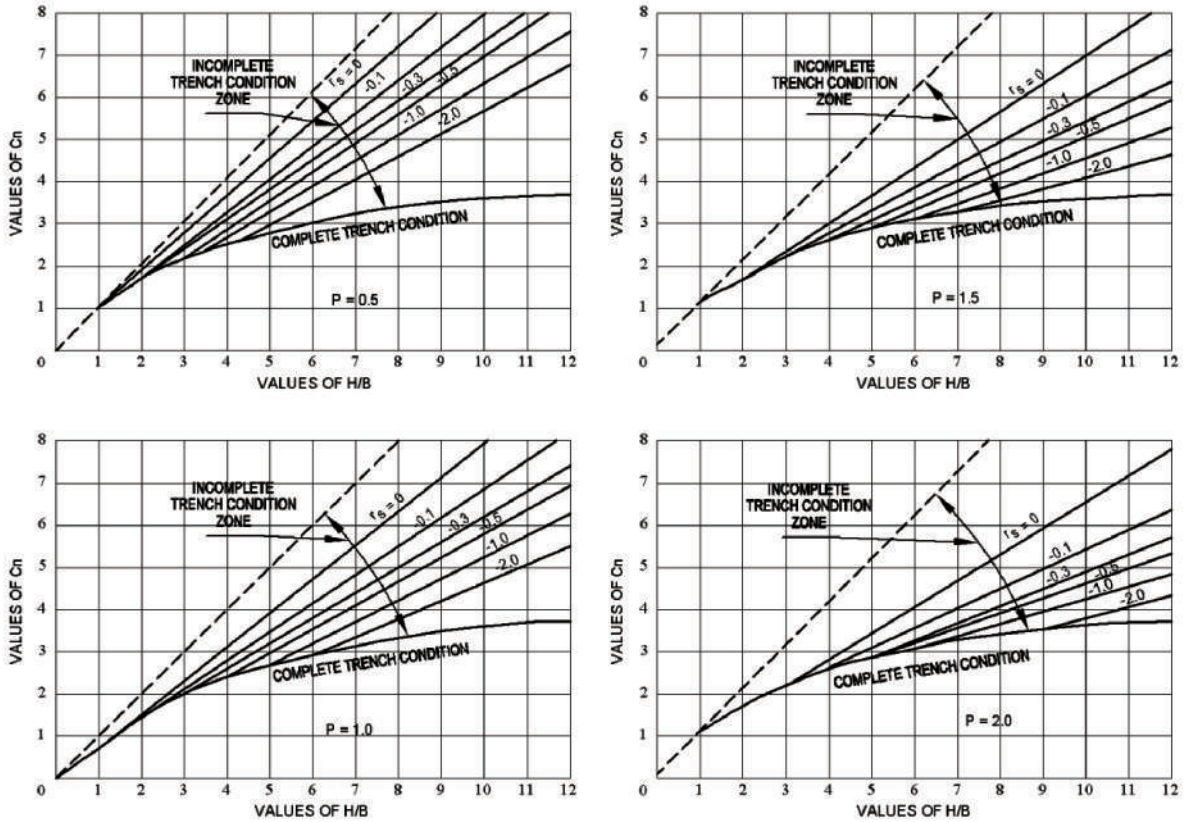
Source: CPHEEO, 1993

Figure 3.36  Coefficient Cn for negative projecting conduits and imperfect trench conditions
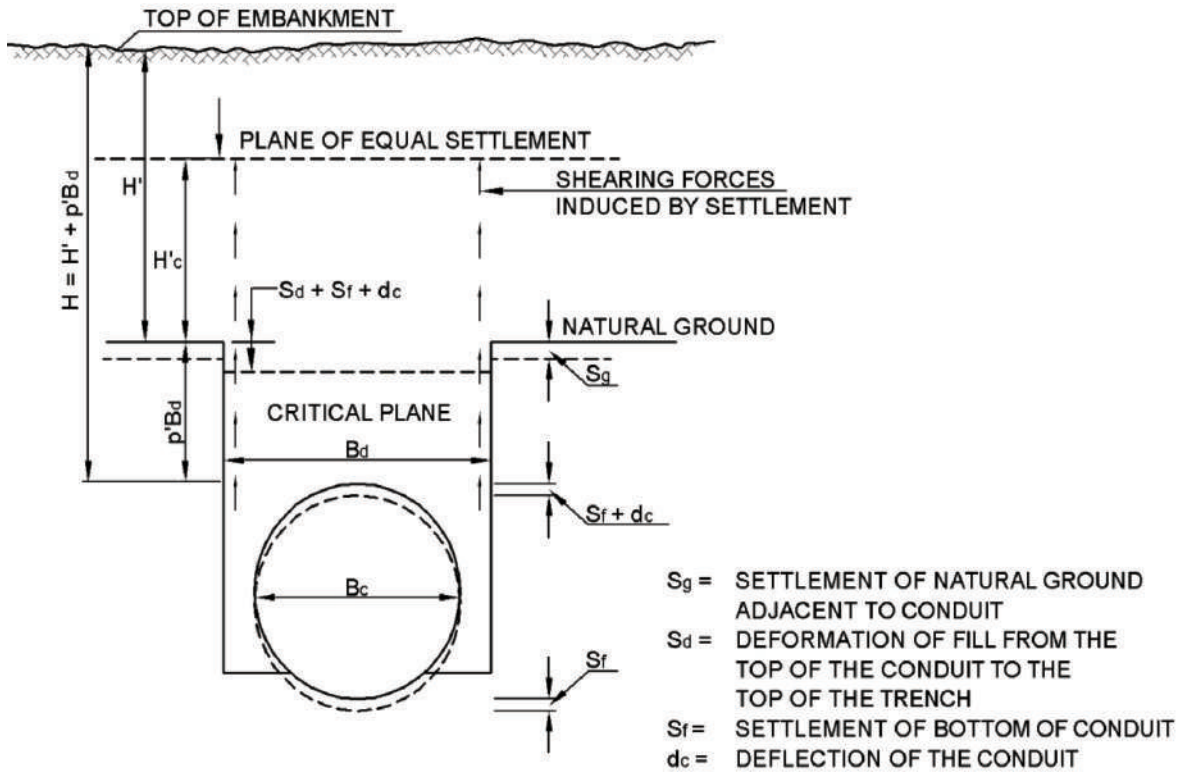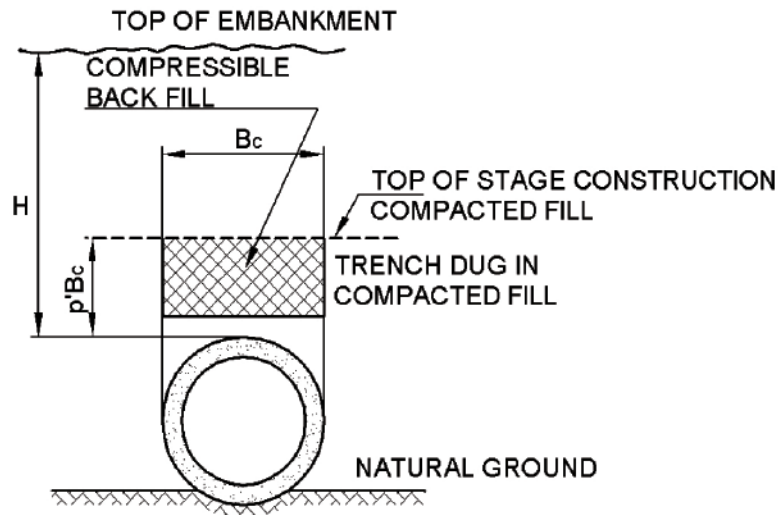


Source: CPHEEO, 1993

Figure 3.37  Settlements that influence loads on negative projecting conduits

c) Imperfect Trench Conduits

An imperfect trench conduit is employed to minimize the load on a conduit under embankments of unusual heights. The conduit is first installed as a positive projecting conduit. The embankment is then built up to some height above the top and thoroughly compacted as it is placed. A trench of the same width as the conduit is excavated directly over it down to or near its top. This trench is refilled with loose compressible material and the balance of the embankment completed in a normal manner (Figure 3.38).



Source: CPHEEO, 1993

Figure 3.38  Imperfect trench conditions

The Marston's formula for this installation condition is again given by

$$W_c = C_n \, w \, B_c{}^2 \qquad\qquad (3.28)$$

The values of Cn in this case also may be obtained from Figure 3.36 for negative projecting conduits taking Bc= Bd on the assumption that the trench fill is no wider than the pipe.

### 3.44    TRENCH CONDITION

Generally, sewers are laid in ditches or trenches by excavation in natural or undisturbed soil and then covered by refilling the trench to the original ground level.

a) Load Producing Forces

The vertical dead load to which a conduit is subjected under trench conditions is the resultant of two major forces. The first component is the weight of the prism of soil within the trench and above the top of the pipe and the second is due to the friction or shearing forces generated between the prism of soil in the trench and the sides of the trench produced by settlement of backfill. The resultant load on the horizontal plane at the top of the pipe within the trench is equal to the weight of the backfill minus these upward shearing forces as shown in Figure 3.39 overleaf.
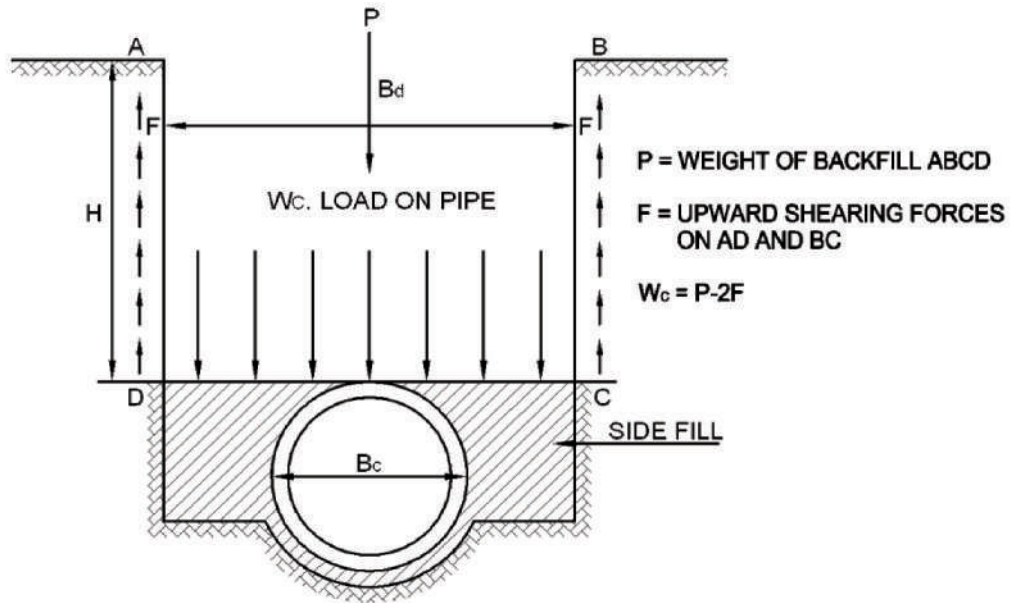
Source: CPHEEO, 1993

Figure 3.39  Load producing forces

b) Computation of Loads

The load on rigid conduits in trench condition is given by the Marston's formula in the form

$$W_c = C_d \, w \, B_d^{\,2}$$ 

(3.29)

where,

$W_c$ :    Load on the pipe in kg per linear meter

$w$ :    Unit weight of backfill soil in kg/m³

$B_d$ :    Width of trench at the top of the pipe in m and

$C_d$ :    Load coefficient which is a function of a ratio of height of fill to width of trench ($H/B_d$) and of the friction coefficient between the backfill and the sides of the trench.

Weights of common filling materials ($w$) and values of $C_d$ for common soil conditions encountered are given in Table 3.18 and Table 3.19, respectively.

Table 3.18  Weights of common filling material

| Materials | Weight (kg/m³) | Materials | Weight (kg/m³) |
|---|---|---|---|
| Dry Sand | 1,600 | Saturated Clay | 2,080 |
| Ordinary (Damp Sand) | 1,840 | Saturated Top Soil | 1,840 |
| Wet Sand | 1,920 | Sand and Damp Soil | 1,600 |
| Damp Clay | 1,920 | | |

Source:  CPHEEO, 1993

Table 3-19  Values of $C_d$ for calculating loads on pipes in trenches

| Ratio $H/B_d$ | Safe working Values of $C_d$ | | | | |
|---|---|---|---|---|---|
| | Minimum possible without cohesion[**] | Maximum for Ordinary Sand[***] | Completely Top Soil | Ordinary maximum for clay[****] | Extreme maximum for clay[*****] |
| 0.5 | 0.455 | 0.461 | 0.464 | 0.469 | 0.474 |
| 1.0 | 0.830 | 0.852 | 0.864 | 0.881 | 0.898 |
| 1.5 | 1.140 | 1.183 | 1.208 | 1.242 | 1.278 |
| 2.0 | 1.395 | 1.464 | 1.504 | 1.560 | 1.618 |
| 2.5 | 1.606 | 1.702 | 1.764 | 1.838 | 1.923 |
| 3.0 | 1.780 | 1.904 | 1.978 | 2.083 | 2.196 |
| 3.5 | 1.923 | 2.075 | 2.167 | 2.298 | 2.441 |
| 4.0 | 2.041 | 2.221 | 2.329 | 2.487 | 2.660 |
| 4.5 | 2.136 | 2.344 | 2.469 | 2.650 | 2.856 |
| 5.0 | 2.219 | 2.448 | 2.590 | 2.798 | 3.032 |
| 5.5 | 2.286 | 2.537 | 2.693 | 2.926 | 3.190 |
| 6.0 | 2.340 | 2.612 | 2.782 | 3.038 | 3.331 |
| 6.5 | 2.386 | 2.675 | 2.859 | 3.137 | 3.458 |
| 7.0 | 2.423 | 2.729 | 2.925 | 3.223 | 3.571 |
| 7.5 | 2.454 | 2.775 | 2.982 | 3.299 | 3.673 |
| 8.0 | 2.479 | 2.814 | 3.031 | 3.366 | 3.764 |
| 8.5 | 2.500 | 2.847 | 3.073 | 3.424 | 3.845 |
| 9.0 | 2.518 | 2.875 | 3.109 | 3.476 | 3.918 |
| 9.5 | 2.532 | 2.898 | 3.141 | 3.521 | 3.983 |
| 10.0 | 2.543 | 2.918 | 3.167 | 3.560 | 4.042 |
| 11.0 | 2.561 | 2.950 | 3.210 | 3.626 | 4.141 |
| 12.0 | 2.573 | 2.972 | 3.242 | 3.676 | 4.221 |
| 13.0 | 2.581 | 2.989 | 3.266 | 3.715 | 4.285 |
| 14.0 | 2.587 | 3.000 | 3.283 | 3.745 | 4.336 |
| 15.0 | 2.591 | 3.009 | 3.296 | 3.768 | 4.378 |
| Very Great | 2.599 | 3.030 | 3.333 | 3.846 | 4.548 |

$W_c$    = load on pipe in kg per linear meter
$C_d$    = Coefficient
w      = Weight of trench filling material in kg/m$^3$
$B_d$    = Width of trench a little below the top of the pipe in meters
*        Ratio of height of fill above top of pipe to width of trench a little below the top of the pipe.
**       These values give the loads generally imposed by granular filling materials before tamping or settling.
***      Use these values as safe for all ordinary cases of sand filling.
****     Thoroughly wet. Use these values as safe for all ordinary cases of clay filling.
*****    Completely   saturated. Use these values only for extremely unfavourable conditions.
Source: CPHEEO, 1993

Equation (3.29) gives the total vertical load due to backfill in the horizontal plane at the top of the conduit as shown in Figure 3.39 if the pipe is rigid. For flexible conduits, the formula may be modified as

$$W_c = C_d \, w \, B_c \, B_d$$

(3.30)

where,

B$_c$ :  Outside width of the conduit in m

c)  Influence of Width of Trench

It has been experimentally seen that when the width of the trench excavated is not more than twice the external width of the conduit, the assumption made in the trench condition of loading holds good. If the width of the trench goes beyond three times the outside dimension of the conduit, it is necessary to apply the embankment condition of loading. In the transition width from B$_d$ = 2B$_c$ to B$_d$ = 3B$_c$ computation of load by both the procedures will give the same results.

In case of excavations with sloping sides (possible in undeveloped areas), the provision of a sub-trench (Figure 3.40) minimizes the load on the pipe by reducing the value of B$_d$.
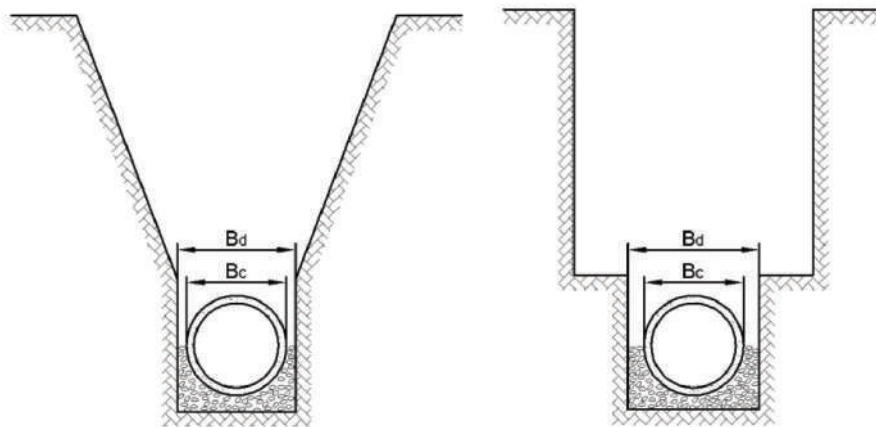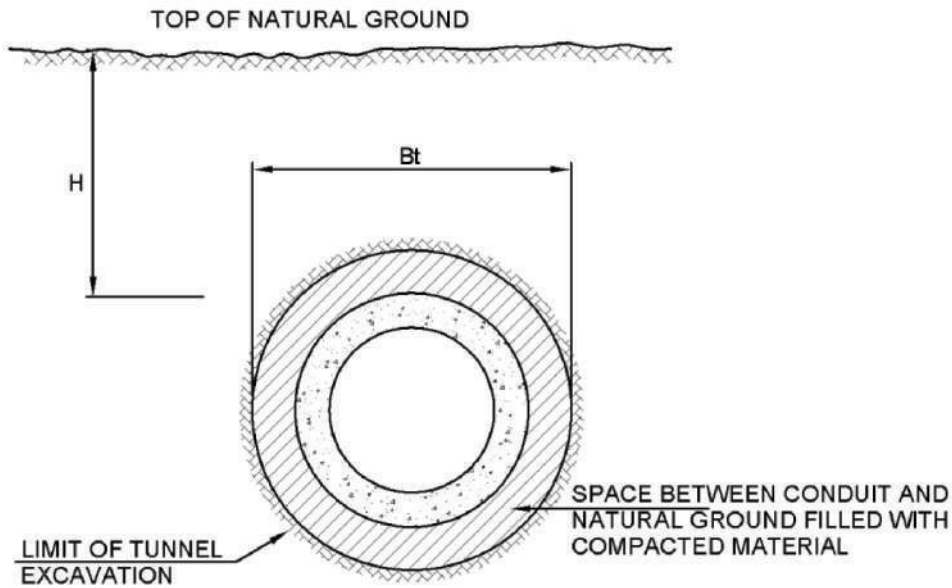


Source: CPHEEO, 1993

Figure 3.40  Examples of sub trench

## 3.45    TUNNEL CONDITION

When the conduit is laid more than 9 m to 12 m deep or when the surface obstructions are such that it is difficult to construct the pipeline by the conventional procedure of excavation and backfilling, it may be more economical to place the conduit by means of tunnelling. The general method in this case is to excavate the tunnel, to support the earth by suitable means and then to lay the conduit. The space between the conduit and the tunnel is finally filled up with compacted earth or concrete grout as indicated in Figure 3.41 overleaf.

If the length of tunnel is short say 6 m to 10 m, the entire circular section can be constructed as one unit. For longer tunnels, construction may be in segments, with refilling proceeding simultaneously.

Source: CPHEEO, 1993

Figure 3.41  Conduit in tunnel

### a)    Load Producing Forces

The vertical load acting on the tunnel supports and eventually the pipe in the tunnel is the resultant of two major forces viz., the weight of the overhead prism of soil within the width of the tunnel excavation and the shearing forces generated between the interior prisms and the adjacent material due to friction and cohesion of the soil.

### b)    Load Computations

Marston's formula to be used in this case of installation of conduit is given by:

$$W_t = C_t\, B_t \left(wB_t - 2C\right) \tag{3.31}$$

where,

$W_t$ :    Load on the pipe or tunnel support in kg/m

$w$ :    Unit weight of soil above the tunnel in kg/m³

$B_t$ :    Maximum width of the funnel excavation in m

$C$ :    Coefficient of cohesion in kg/m² and

$C_t$ :    Load coefficient which is a function of the ratio (H/Bt) of the distance from the ground surface to the top of the tunnel to the maximum width of tunnel excavation and of the coefficient of internal friction of the material of the tunnel.

When the coefficient of cohesion is zero, the formula reduces to the same form as in trench condition equation (3.29).

Value of C for various values of H/Bt and different soil conditions are to be obtained from Figure 3-42.
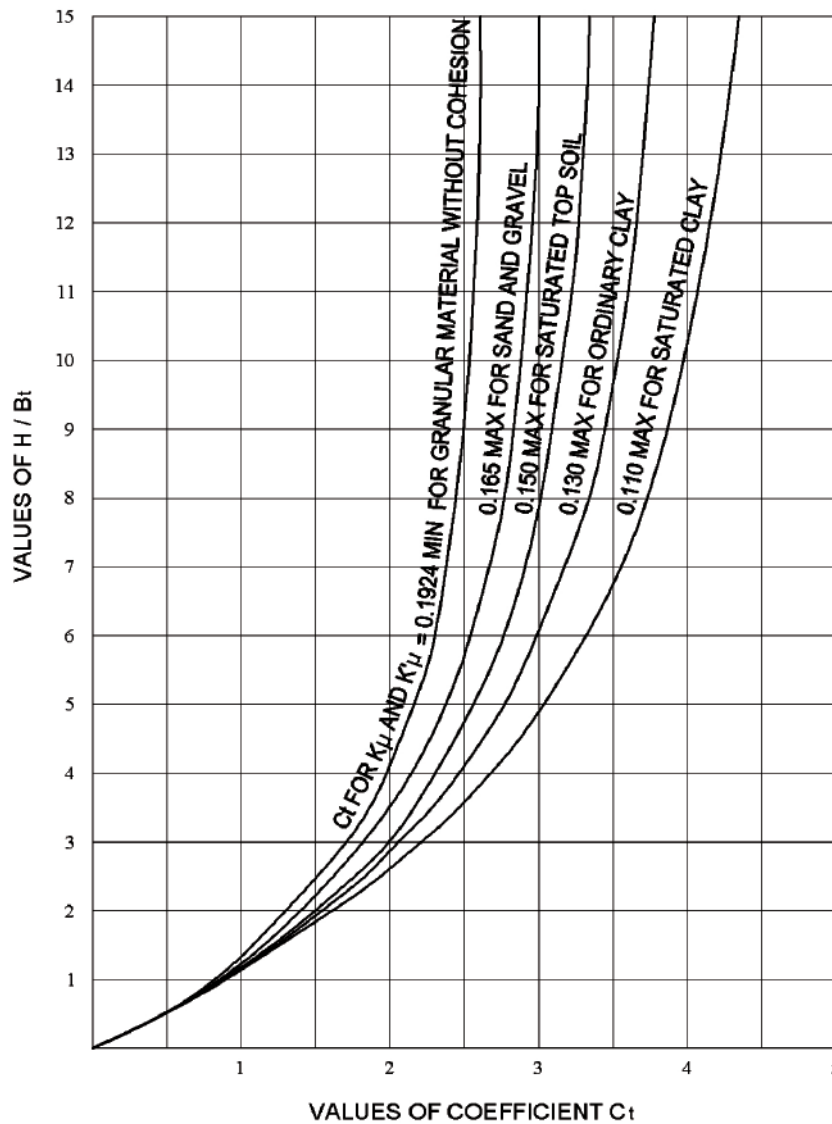


Source: CPHEEO, 1993

Figure 3.42  Diagram for coefficient $C_t$ for tunnels in undisturbed soil

Recommended values of coefficient of cohesion for different types of soils are given in Table 3.20.

Table 3.20  Cohesion coefficients for different soils

| Type of Soil | kg/m$^2$ | Type of Soil | kg/m$^2$ |
|---|---|---|---|
| Soft Clay | 200 | Silty Sand | 500 |
| Medium Clay | 1,200 | Dense Sand | 1,400 |
| Hard Clay | 4,700 | Saturated Top Soil | 500 |
| Loose Dry Sand | 0 | | |

Source: CPHEEO, 1993

## 3.46    EFFECT OF SUBMERGENCE

Sewers may be laid in trenches or under embankment in areas that may be temporarily or permanently submerged in water. The fill load, in such cases, will be reduced and correspond to the buoyant weight of the fill material. However, effect of submergence could be ignored which provides an additional factor of safety, but it may be necessary to check whether a pipe is subject to flotation. Under submergence, the minimum height of the fill material that will be required to prevent flotation ignoring the frictional forces in the fill can be determined from the equation:

$$H_{min}\ B_c\ (w_s - w_o) + W_c = (\pi/4)\ B_c^{\ 2}\ w_o \qquad\qquad (3.32)$$

where,

$H_{min}$ :    Minimum height of fill material in m

$w_s$ :    Saturated density of the soil in $kg/m^3$

$w_0$ :    Density of water in $kg/m^3$

$W_c$ :    Unit weight of the empty pipe in kg/linear metre and

$B_c$ :    Outside width of the conduit in m.

Wherever sufficient height of fill material is not available, anti-flotation blocks should be provided. (As shown in Example IX in Appendix A.3.8)

### 3.46.1    Load on Conduit due to Superimposed Loads

The types of superimposed loads, which are generally encountered in buried conduits may be categorized as (a) concentrated load and (b) distributed load. These are explained diagrammatically in Figure 3.43.
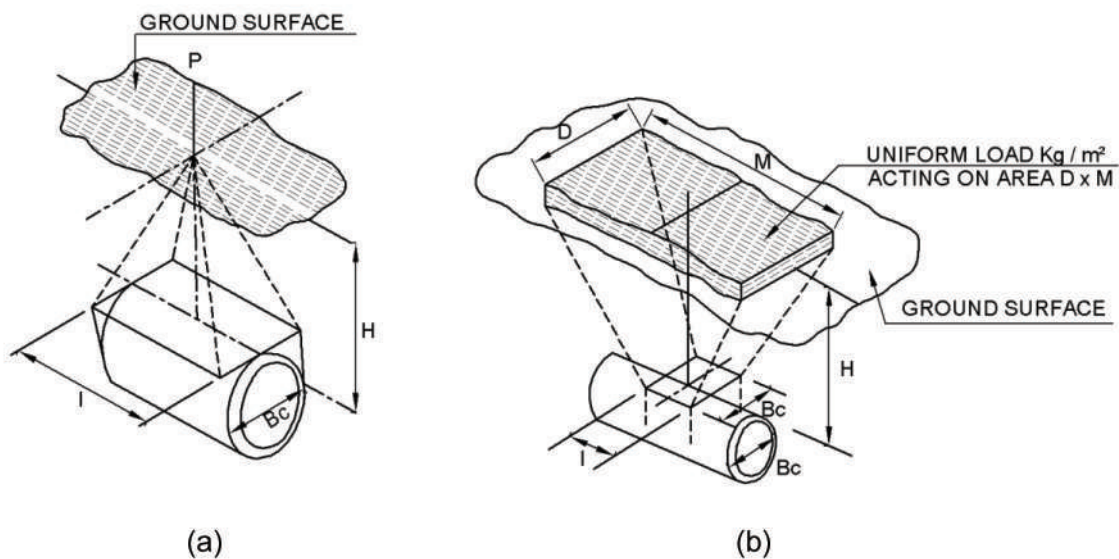


Source:CPHEEO, 1993

Figure 3.43  (a) Concentrated superimposed load vertically centred over conduit (left)
(b) Distributed superimposed load vertically centred over conduit (right)

## 3.47   CONCENTRATED LOADS

The formula for load due to superimposed concentrated load such as a truck wheel (Figure 3.43) is given in the following form by Holl's integration of Boussinesq's formula

$$W_{sc} = C_s \left( PF / L \right)$$   (3.33)

where,

$W_{sc}$ :   Load on the conduit in kg/m

$P$ :   Concentrated load in kg acting on the surface

$F$ :   Impact factor (1.0 for air field runways, 1.5 for highway traffic and air field taxi ways, 1.75 for railway traffic) and

$C_s$ :   Load coefficient which is a function of

$$\frac{B_c}{2H} \quad and \quad \frac{L}{2H}$$   (3.34)

where,

$H$ :   Height of the top of the conduit to ground surface in m

$B_c$ :   Outside width of conduit in m, and

$L$ :   Effective length of the conduit to which the load is transmitted in m.

Values of Cs for various values of ($B_c/2H$) and ($L/2H$) are obtained from Table 3.20

The effective length of the conduit is defined as the length over which the average load due to surface traffic units produces the same stress in the conduit wall, as does the actual load, which varies in intensity from point to point. This is generally taken as 1 m or the actual length of the conduit if it is less than 1 m.

## 3.48   DISTRIBUTED LOAD

For the case of distributed superimposed loads, the formula for load on conduit is given by

$$W_{sd} = C_s \, p \, F \, B_c$$   (3.35)

where,

$W_{sd}$ :   Load on the conduit in kg/m

$P$ :   Intensity of the distributed load in kg/m$^2$

$F$ :   Impact factor

$B_c$ :   Width of the conduit in m

$C_s$ :   Load coefficient, a function of $D/2H$ and $L/2H$ from Table 3.21

$H$ :   Height of the top of conduit to the ground surface in m

$D, L$ :   Width and length in m respectively of the area over which the distributed load acts

Table 3.21  Values of load coefficients, Cs for concentrated and distributed superimposed loads vertically centred over conduits

| $\dfrac{D}{2H}$ or $\dfrac{B_c}{2H}$ | $\dfrac{M}{2H}$ or $\dfrac{L}{2H}$ | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.1 | 0.2 | 0.3 | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 | 0.9 | 1.0 | 1.2 | 1.5 | 2.0 | 5.0 |
| 0.1 | 0.019 | 0.037 | 0.053 | 0.067 | 0.079 | 0.089 | 0.097 | 0.103 | 0.108 | 0.112 | 0.117 | 0.121 | 0.124 | 0.128 |
| 0.2 | 0.037 | 0.072 | 0.103 | 0.131 | 0.155 | 0.174 | 0.189 | 0.202 | 0.211 | 0.219 | 0.229 | 0.238 | 0.244 | 0.248 |
| 0.3 | 0.053 | 0.103 | 0.149 | 0.190 | 0.224 | 0.252 | 0.274 | 0.292 | 0.306 | 0.318 | 0.333 | 0.345 | 0.355 | 0.360 |
| 0.4 | 0.067 | 0.131 | 0.190 | 0.241 | 0.284 | 0.320 | 0.349 | 0.373 | 0.391 | 0.405 | 0.425 | 0.440 | 0.454 | 0.460 |
| 0.5 | 0.079 | 0.155 | 0.224 | 0.284 | 0.336 | 0.379 | 0.414 | 0.441 | 0.463 | 0.481 | 0.505 | 0.525 | 0.540 | 0.548 |
| 0.6 | 0.089 | 0.174 | 0.252 | 0.320 | 0.379 | 0.428 | 0.467 | 0.499 | 0.524 | 0.544 | 0.572 | 0.596 | 0.613 | 0.624 |
| 0.7 | 0.097 | 0.189 | 0.274 | 0.349 | 0.414 | 0.467 | 0.511 | 0.546 | 0.584 | 0.597 | 0.628 | 0.650 | 0.674 | 0.688 |
| 0.8 | 0.103 | 0.202 | 0.292 | 0.373 | 0.441 | 0.499 | 0.546 | 0.584 | 0.615 | 0.639 | 0.674 | 0.703 | 0.725 | 0.740 |
| 0.9 | 0.108 | 0.211 | 0.306 | 0.391 | 0.463 | 0.524 | 0.574 | 0.615 | 0.647 | 0.673 | 0.711 | 0.742 | 0.766 | 0.784 |
| 1.0 | 0.112 | 0.219 | 0.318 | 0.405 | 0.481 | 0.544 | 0.597 | 0.639 | 0.673 | 0.701 | 0.740 | 0.774 | 0.800 | 0.816 |
| 1.2 | 0.117 | 0.229 | 0.333 | 0.425 | 0.505 | 0.572 | 0.628 | 0.674 | 0.711 | 0.740 | 0.783 | 0.820 | 0.849 | 0.868 |
| 1.5 | 0.121 | 0.238 | 0.345 | 0.440 | 0.525 | 0.596 | 0.650 | 0.703 | 0.742 | 0.774 | 0.820 | 0.861 | 0.894 | 0.916 |
| 2.0 | 0.124 | 0.244 | 0.355 | 0.454 | 0.540 | 0.613 | 0.674 | 0.725 | 0.766 | 0.800 | 0.849 | 0.894 | 0.930 | 0.956 |

Source: CPHEEO, 1993

For class AA IRC loading, in the critical case of wheel load of 6.25 tones, the intensity of distributed load with wheel area 300mm × 150mm is given by

$$P = \frac{6.25}{0.3 \times 0.15} \quad in \quad T/m^2$$

## 3.49    CONDUITS UNDER RAILWAY TRACK

The load on conduits under railway track is given by

$$W = 4\,C_s\,U\,B_c \tag{3.36}$$

where,

U: Uniformly distributed load in tonnes / m2 from the surface directly over the conduit and equal to

$$U = \frac{PF + 2W_t B}{4AB} = \frac{PF}{4AB} + \frac{W_t}{2A} \tag{3.37}$$

where,

$P$ :    Axle load in tonnes (22.5 tonnes for Broad gauge)

$F$ :    Impact factor for railroad =1.75

$2A$ :    Length of the sleeper in m (2.7 m for Broad gauge)

$2B$ :    Distance between the two axles (1.84 m for broad gauge)

$W_t$ :    Weight of the track structure in tones/m (0.3 tonnes/m for broad gauge)

$C_s$ :    Load coefficient which depends on the height of the top of sleeper from the top of the conduit

$B_c$ :    Width of the conduit in m

For broad gauge track, the formula will reduce to:

$$W = 32.14\,C_s\,B_c \tag{3.38}$$

## 3.50    SUPPORTING STRENGTH OF RIGID CONDUIT

The ability of a conduit to resist safely the calculated earth load depends not only on its inherent strength but also on the distribution of the vertical load and bedding reaction and on the lateral pressure acting against the sides of the conduit. The inherent strength of a rigid conduit is usually expressed in terms of the three edge bearing test results, the conditions of which are, however, different from the field load conditions. The magnitude of the supporting strength of a pipe as installed in the field is dependent upon the distribution of the vertical load and the reaction against the bottom of the pipe. It also depends on the magnitude and distribution of the lateral pressure acting on the sides of the pipe.

### 3.50.1 Laboratory Test Strength

All rigid pipes may be tested for strength in the laboratory by the three-edge-bearing test (ultimate load). Methods of test and minimum strength for concrete (unreinforced and reinforced) stoneware and AC pipes and other details are given in Appendix A.3.9.

### 3.50.2 Field Supporting Strength

The field supporting strength of a rigid conduit is the maximum load per unit length, which the pipe will support while retaining complete serviceability when installed under specified conditions of bedding and backfilling. The field supporting strength however does not include any factor of safety. The ratio of the strength of a pipe under any stated condition of loading and bedding to its strength measured by the three-edge-bearing test is called the load factor.

The load factor does not contain a factor of safety. Load factors have been determined experimentally and analytically for the commonly used construction condition for both trench and embankment conduits. The basic design relationship between the different design elements is the safe supporting strength (W),

W = Field supporting strength / Factor of safety

= Load factor × three edge bearing strength / Factor of safety          (3.39)

A factor of safety of at least 1.5 should be applied to the specified minimum three-edge-bearing strength to determine the working strength for all the rigid conduits.

### 3.50.3 Protection and Bedding of Sewers

### 3.50.3.1 Guidelines

The factor of safety recommended for concrete pipes for sewers is '1.5', which is considerably less as compared to that for most engineering structures which have a factor of safety of at least 2.5. As the margin of safety against the ultimate failure is low, it becomes imperative to guarantee that the loads imposed on sewer pipes are not greater than the design loads for the given installation conditions. In order to achieve this objective the following procedures are recommended:

1. Minimum width of the trench should be specified in consonance with the requirements of adequate working space to allow access to all parts and joints of pipes.

2. Specification should lay proper emphasis on the limit of the width of trench to be adopted in the field, which should not exceed that adopted in the design calculations. Any deviations from this requirement during the construction should be investigated for their possible effect on the load coming on the pipe and steps should be taken to improve the safe supporting strength of pipe for this condition of loading by adopting suitable bedding or such other methods when necessary.

3. The field engineer should keep in touch with the design engineer throughout the duration of the project and any deviation from the design assumptions due to the exigencies of work, should be immediately investigated and corrective measures taken in time.

4. All pipes used on the work should be tested as per the IS specifications and test certificates of the manufacturers should be furnished for every consignment brought to the site.

5. Whenever shoring is used, the pulling out of planks on completion of work should be carried out in stages and this should be properly supervised to ensure that the space occupied by the planks is properly backfilled.

6. Proper backfilling methods both as regards to selection of materials, methods of placing and proper compaction should be in general agreement with the design assumptions.

### 3.50.3.2 Bedding in Quicksand Soil Conditions

In quick sand conditions, it is necessary to anchor the sewer to the ground and hold it at the grade as laid in the face of soil sinkage. This is done by using the Venteak piles, which are driven on both sides of the sewer into the soil right up to hard strata and connecting the two by a cross beam at the soffit of the sewer. Then the sewer is tied securely to the cross beam by a 8 mm thick nylon rope in two rounds and singeing the ends of the rope integrally to prevent slippages. An example is shown in Figure 3-44. The venteak pile cross bracing can be a single brace inserted between the piles for non-metallic smaller sewers and double bracing for metallic higher sized sewers as in Figure 3.44. A work in progress in such conditions is shown in Figure 3.45 overleaf.
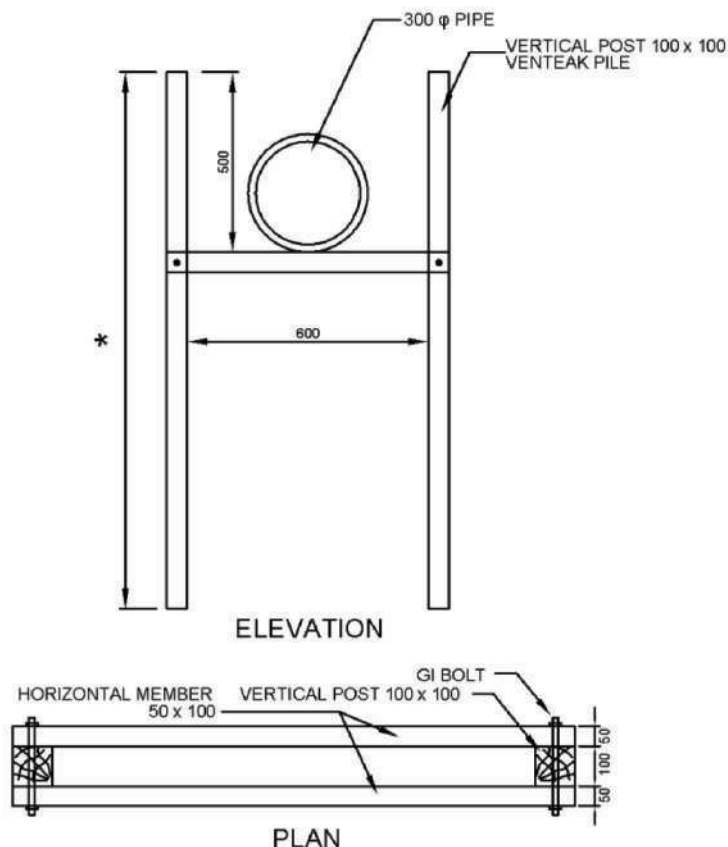


(The sewer pipe should be cross-braced with the horizontal supports by means of non-biodegradable nylon rope of 8 mm multi-stranded and with multiple wraps around and the edges singed to heat weld the entire rope without loosening or unwinding)

Figure 3.44  Example of Venteak supported sewer pipe

Source: L&T Shipbuilding Limited, Kattupalli, Chennai, 2012

Figure 3.45 Typical arrangements for laying sewers in high subsoil locations using dewatering pump sets, tube wells and Venteak piles with cross brace and nylon rope wrapping around the sewers securing it to the venteak piles and brace

### 3.50.3.3    Type of Bedding

The type of bedding (granular, concrete cradle, full concrete encasement etc.) would depend on the soil strata and depth at which sewer 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.

$$\text{Bedding Factor} \;=\; \frac{\text{Design Load X Factor of Satety*}}{\text{Three Edge Bearing Strength}} \qquad (3.40)$$

* 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 Table 3.22 and Table 3.23.

Table 3.22  Type of bedding for sewer pipes

| 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 3.23  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 | 1,400 | B | Ab | Ab | Ab |
| 500 | Ab | Ab | Ab | Ab | 1,500 | B | Ab | Ab | Ab |
| 600 | B | Ab | Ab | Ab | 1,600 | B | Ab | Ab | Ab |
| 700 | B | Ab | Ab | Ab | 1,800 | B | Ab | Ab | Ab |
| 750 | B | Ab | Ab | Ab | 2,000 | B | Ab | Ab | Ab |
| 800 | B | Ab | Ab | Ab | 2,200 | B | Ab | Ab | Ac |
| 900 | B | Ab | Ab | Ab | 2,400 | B | Ab | Ab | Ac |
| 1,000 | B | Ab | Ab | Ab | 2,600 | B | Ab | Ab | Ac |
| 1,200 | B | Ab | Ab | Ab | 2,800 | B | Ab | Ab | Ac |

### 3.50.3.4    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 3-46 overleaf. 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.
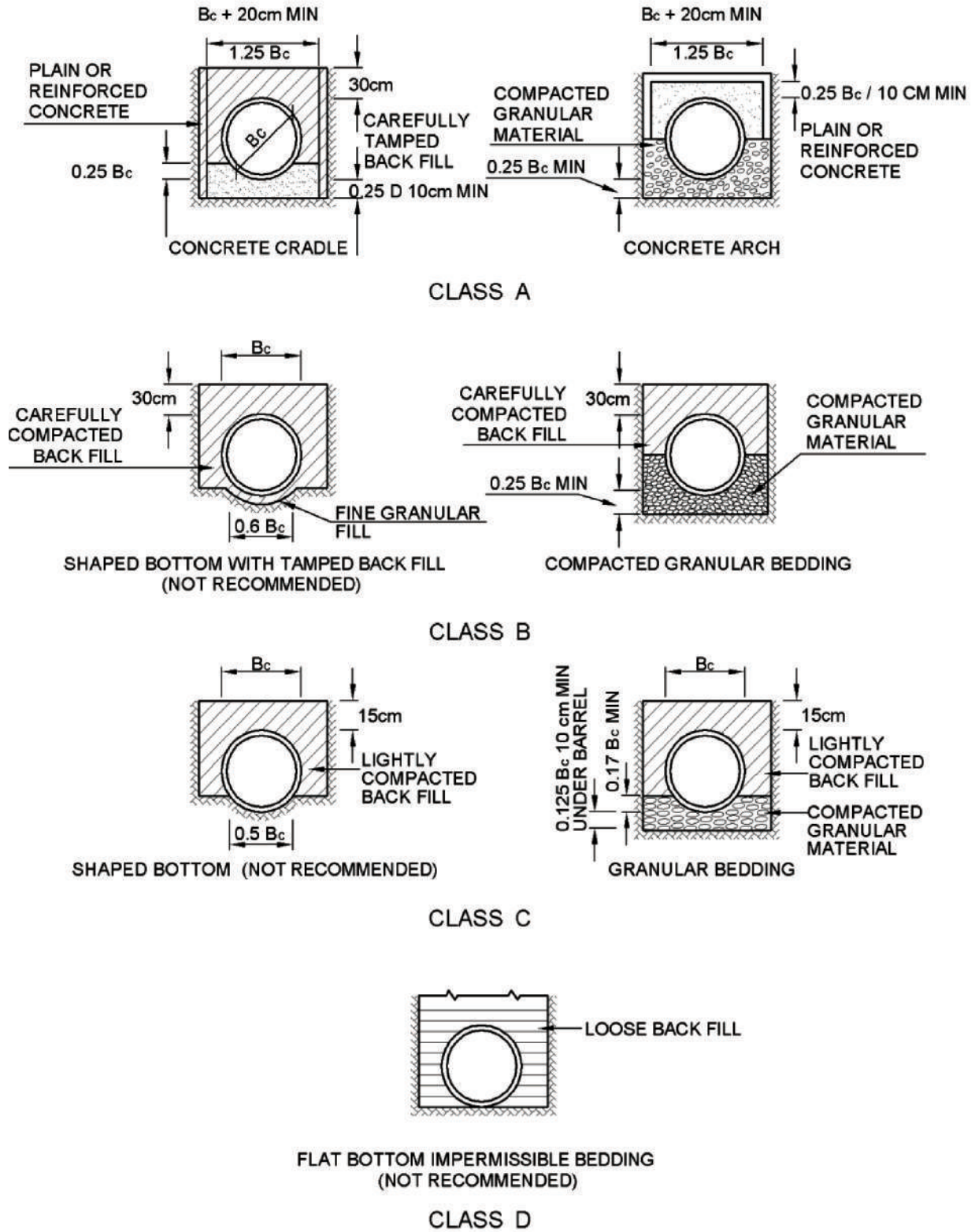
The pipe bedding materials must remain firm and not permit displacement of pipes.

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 sewer pipes.

PLAIN OR REINFORCED CONCRETE

Bc + 20cm MIN

1.25 Bc

30cm

CAREFULLY TAMPED BACK FILL

0.25 Bc

0.25 D 10cm MIN

CONCRETE CRADLE

Bc + 20cm MIN

1.25 Bc

COMPACTED GRANULAR MATERIAL

0.25 Bc / 10 CM MIN

PLAIN OR REINFORCED CONCRETE

0.25 Bc MIN

CONCRETE ARCH

CLASS A

Bc

30cm

CAREFULLY COMPACTED BACK FILL

FINE GRANULAR FILL

0.6 Bc

SHAPED BOTTOM WITH TAMPED BACK FILL
(NOT RECOMMENDED)

CAREFULLY COMPACTED BACK FILL

30cm

COMPACTED GRANULAR MATERIAL

0.25 Bc MIN

COMPACTED GRANULAR BEDDING

CLASS B

Bc

15cm

LIGHTLY COMPACTED BACK FILL

0.5 Bc

SHAPED BOTTOM (NOT RECOMMENDED)

0.125 Bc 10 cm MIN UNDER BARREL

0.17 Bc MIN

Bc

15cm

LIGHTLY COMPACTED BACK FILL

COMPACTED GRANULAR MATERIAL

GRANULAR BEDDING

CLASS C

LOOSE BACK FILL

FLAT BOTTOM IMPERMISSIBLE BEDDING
(NOT RECOMMENDED)

CLASS D

NOTE : IN ROCK, TRENCH IS EXCAVATED AT LEAST 15cm
BELOW THE BELL OF THE PIPE EXCEPT WHERE
CONCRETE CRADLE IS USED.

Source: CPHEEO, 1993

Figure 3.46  Classes of bedding for conduit in trench

### 3.50.3.5    Load Factors for Bedding

The load factors for the different classes of bedding are given in Table 3-24.

Table 3.24  Load factor for different classes of bedding

| Class of bedding | Condition | Load factor |
|---|---|---|
| Aa | Concrete cradle-plain concrete and lightly tamped backfill | 2.2 |
| Ab | Concrete cradle-plain concrete and carefully tamped backfill | 2.8 |
| Ac | Concrete cradle-RCC with P-0.4% | Up to 3.4 |
| Ad | Arch type – plain concrete | 2.8 |
| | RCC with P = 0.4% | Up to 3.4 |
| | RCC with P = 0.1% | Up to 4.8 |
| | P  is ratio of area of steel to area of concrete at the crown) | |
| B | Shaped bottom or compacted granular bedding with carefully compacted backfill | 1.9 |
| C | Shaped bottom or compacted granular bedding with lightly compacted backfill | 1.5 |
| D | Flat bottom trench | 1.1 |

Source:CPHEEO, 1993

The granular material used must stabilize the trench bottom in addition to providing a firm and uniform support for the pipe. Well-graded crushed rock or gravel with the maximum size not exceeding 25 mm, is recommended for the purpose. Where rock or other unyielding foundation material is encountered, bedding may be according to one of the Classes A, B or C, but with the following additional requirements.

**Class A:**   The hard unyielding material should be excavated down to the bottom of the concrete cradle.

**Class B or Class C:**   The hard, unyielding material should be excavated below the bottom of the pipe and pipe bell, to a depth of at least 15 cm.

The width of the excavation should be at least 1.25 times the outside diameter of the pipe and it should be refilled with granular material.

Total encasement of non-reinforced rigid pipe in concrete may be necessary where the required safe supporting strength cannot be obtained by other bedding methods.

The load factor for concrete encasement varies with the thickness of concrete. The effect of M-20 concrete encasement of various thicknesses on supporting strength of pipe under trench conditions is given in Figure 3.47 overleaf.
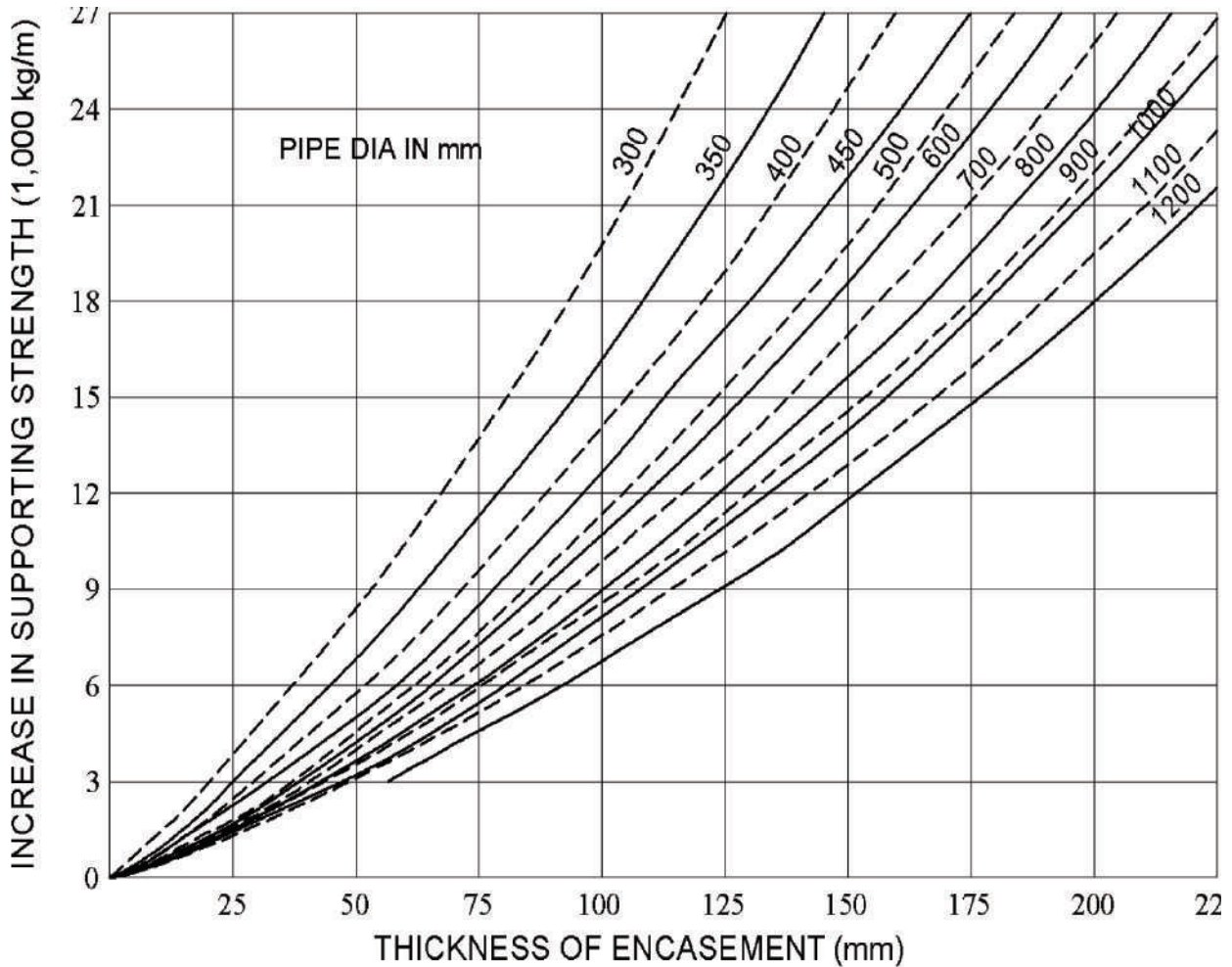
Source: CPHEEO, 1993

Figure 3.47  Effect of M-20 concrete encasement of various thickness on supporting strength of
pipe under trench conditions

### 3.50.3.6    Supporting Strength in Embankment Conditions

The soil pressure against the sides of a pipe placed in an embankment may be significant in resisting
the vertical load on the structure.

### 3.50.3.7    Classes of Bedding

The beddings, which are generally adopted for projecting conduits laid under the  embankment conditions
of installation are illustrated in Figure 3.48 overleaf.

The classifications of the beddings are as under:

**CLASS A:**    In this case, the conduit is laid on a mat of concrete.

**CLASS B:**    The conduit is laid on accurately shaped earth to fit the bottom of the pipe and the
sides are filled with thoroughly tamped earth.
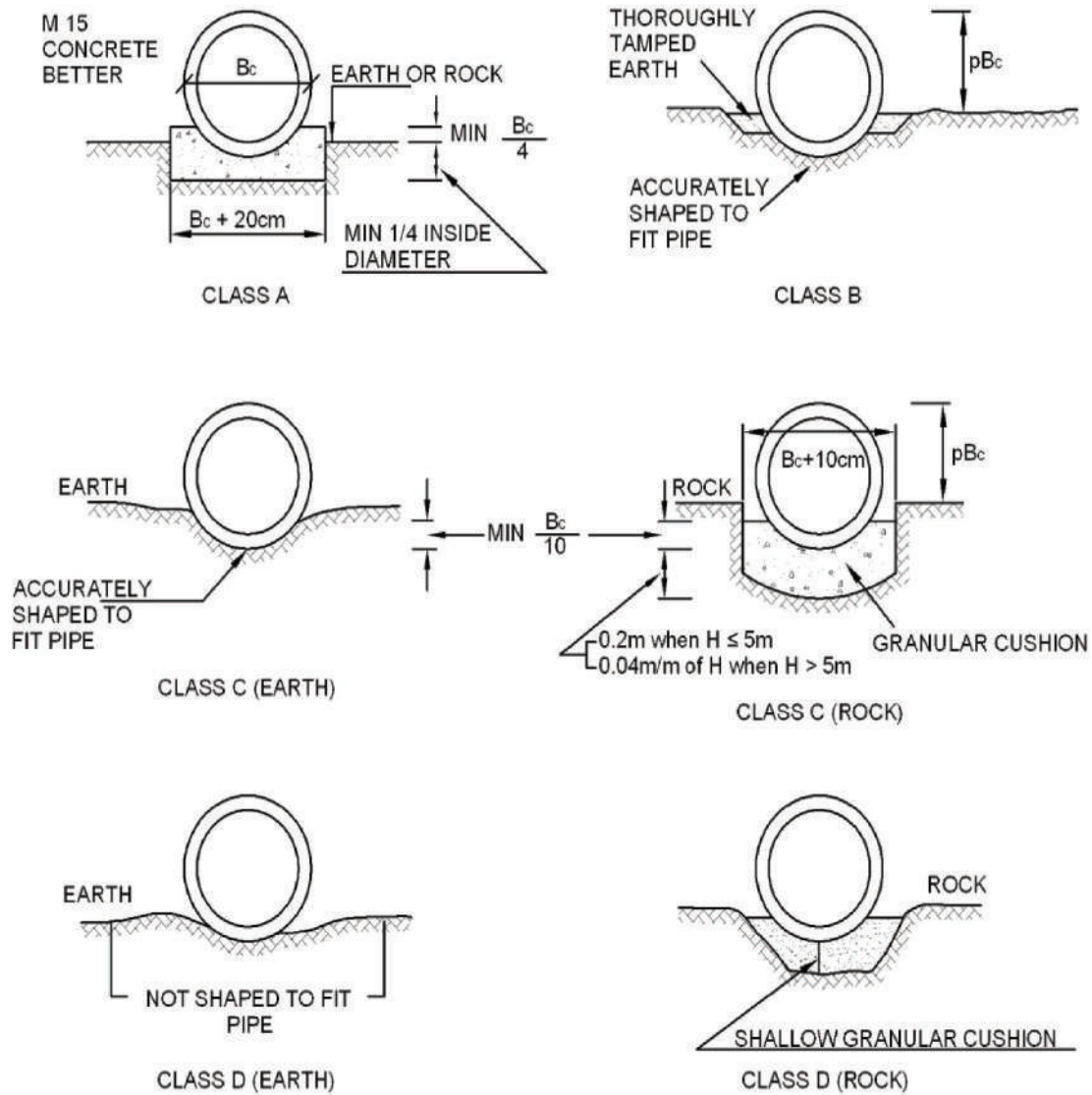
Source: CPHEEO, 1993

Figure 3.48  Classes of bedding for projecting conduits

**CLASS C:**     In this type of bedding the conduit is laid on accurately shaped earth to fit the bottom surface of the conduit. For rock foundations, the conduit is laid on a layer of granular cushion and the sides of the conduit are filled up.

**CLASS D:**     The conduit is laid on earth not shaped to fit the bottom of the conduit. In case of rocky soil, the conduit is laid on a shallow granular cushion.

### 3.50.3.8    Load Factors for Rigid Pipes

The load factor for rigid pipes, installed as projecting conduits under embankments or in wide trenches, is dependent on the type of bedding, the magnitude of the active lateral soil pressure and on the area of the pipe over which the active lateral pressure acts.

The load factor for projecting circular conduits may be calculated by the formula:

$$L_f = \frac{1.431}{NZq} \tag{3.41}$$

where,

$L_f$ : Load factor

$N$ : Parameter dependent on the type of bedding

$Z$ : Parameter dependent upon the area over which the lateral pressure acts effectively

$q$ : Ratio of total lateral pressure to total vertical load on pipe

## a) Positive Projecting Conduits

The ratio q for positive projecting conduits may be estimated by the formula

$$q = (mk/C_c)\left[(H/B_c)+(m/2)\right] \tag{3.42}$$

where,

k : Rankine's ratio which may be taken as 0.33.

The value of N for different types of beddings for circular pipes is given in Table 3.25.

Table 3-25  Values of N for different pipe beddings

| Type of Bedding | Value of 'N' | Type of Bedding | Value of 'N' |
|---|---|---|---|
| 'A' - Reinforced concrete cradle | 0.42 to 0.51 | 'C' | 0.84 |
| 'A' - Plain concrete cradle | 0.51 to 0.64 | 'D' | 1.31 |
| 'B' | 0.71 | | |

Source: CPHEEO, 1993

The value of Z in case of circular pipes is given in Table 3.26.

Table 3.26  Values of Z for different pipe beddings

| Fraction of conduit on which lateral pressure act 'm' | Value of 'Z' for | | Fraction of conduit on which lateral pressure act 'm' | Value of 'Z' for | |
|---|---|---|---|---|---|
| | 'A' Class Beddings | Other Beddings | | 'A' Class Beddings | Other Beddings |
| 0.0 | 0.150 | 0.000 | 0.7 | 0.811 | 0.594 |
| 0.3 | 0.743 | 0.217 | 0.9 | 0.678 | 0.655 |
| 0.5 | 0.856 | 0.423 | 1.0 | 0.638 | 0.638 |

Source:CPHEEO, 1993

## b) Negative Projecting Conduits

The load factor for negative projecting conduits may also be determined by the equations (3.41) and (3.42) with a value of k of 0.15, provided the side fills are well compacted.

## c) Imperfect Trench Conditions

The equations for positive projecting conditions will hold good for those conditions as well.

### 3.50.3.8.1 Conduits under Simultaneous Internal Pressure and External Loading

Simultaneous action of internal pressure and external load gives a lower supporting strength of a pipe than what it would be if the external load acted alone.

If the bursting strength and the three-edge strength of a pipe are known, the relation between the internal pressure and external loads, which will cause failure may be computed by means of the formula:

$$t = \frac{T\left(1 - s^2\right)}{S}$$  (3.43)

where,

t :    Internal pressure in kg/cm$^2$ at failure when external load is simultaneously acting

T :    Bursting strength of a pipe in kg/cm$^2$ when no external load is simultaneously acting

s :    Three-edge-bearing load at failure in kg/linear metre when there is simultaneous action of internal pressure and

S :    Three-edge-bearing load at failure in kg/linear metre when there is no internal pressure simultaneously acting.

## 3.51   RELATIONSHIP BETWEEN DIFFERENT ELEMENTS IN STRUCTURAL DESIGN

The basic design relationships between the different design elements are as follows for rigid pipe

$$\text{Safe working strength} = \frac{\text{Ultimate three edge bearing strength}}{\text{Factor of safety}}$$  (3.44)

Safe field supporting strength = Safe working strength × load factor   (3.45)

Appendix A.3.9 gives the details of three-edge-bearing tests.

It is but obvious, that sewers have to be sturdy enough to sustain the load of the backfill material (dead load), as well as the load due to the vehicular traffic (live load). Factors like, depth of the backfill, type of this material, and width of the trench influence the magnitude of the dead load; while the parameters that determine the load-carrying capacity of the sewer line are the crushing strength of the pipe, and the characteristics of the pipe bedding. Bedding defines the way in which a pipe is placed on the bottom of the trench.

Proper bedding distributes the load around the circumference of the pipe, and this increases the supporting strength of the pipe. The ratio of actual field supporting strength to the crushing strength of the pipe is known as load factor.

It may be pointed out that class D bedding is the weakest of all, and hence is not generally adopted. Here, the trench bed being left flat and bare, the pipe is not fully supported due to its projecting bell-ends. Further, if the backfill is placed loosely over the sewer without the necessary compaction, the barrel may not get properly supported by the bedding. The ordinary bedding (Class C), offers a better support, say, with a load factor of 1.5. In first class bedding (Class B), the granular material extends halfway up the pipe, and a carefully compacted backfill can give a load factor of even 1.9. In Class A bedding, the barrel is supported by a concrete bed (yielding a load factor of 2.8) with a careful compaction of the backfill. It is common, in such engineering constructions to define a safely factor (SF) as well, such as:

$$\text{Safety Factor} = \frac{\text{Field Supporting Strength}}{\text{Safe Supporting Strength}} \qquad (3.46)$$

Safety factor of 1.5 is normally adopted for clay or unreinforced concrete sewers to address the possibility of using poor quality materials or for faulty construction. With a view to selecting the best bedding condition, it is to be ensured that the safe supporting strength is equal to or greater than the total expected load over the pipe.

For pipelines situated in shallower trenches (such as, storm sewers or even some water mains), the component of load due to vehicular traffic may be a substantial part of the total load on the line. However, for deeper trenches (such as, sanitary sewers), the proportion of live load may not be significant compared to the dead load. In USA, Marston's Formula is commonly used to determine the load due to backfill, as in Equation (3.23).

### 3.51.1 Field Layout and Installation

It is understood that the straight line and slope of a sewer has to be carried out meticulously as per design. The horizontal layout determines the location as well as direction of the sewer line, while slope of the line provides the necessary hydraulic carrying capacity of the sewerage system.

The location of the trench is generally laid out first as an offset line running parallel to the proposed sewer centre line. This offset line is demarcated by wooden stakes driven into the ground surface at intervals of, say, 15 m. The offset line, as is clear, is quite away from the sewer centre line with a view not to allow it being disturbed during construction; however, it has to be proximate enough so that the transfer of measurements to the actual trench can readily be done. The wooden stakes are set with their tops at a specific height above the designed trench bottom (horizontal slope line) thus, the checking of the trench depth during excavation, etc., can be done with ease.

Two procedures are available to lay pipe sections in the open trench, namely, by batter boards, and by laser beams. Batter boards are placed across the trench at uniform intervals. The tops of these boards can be set at even height above the designed sewer invert elevation.

The centre line of the sewer is traced on the boards by extending a line of sight with a transit level or a theodolite and a string is stretched from board to board along this very line. Later on, this line is transferred onto the trench bed by means of a plumb bob for the invert levels. Invert levels and characteristics indicated by vertical rods are marked off in even increments and the lower end of each rod is placed on the pipe invert bedding plane, and the string over the batter boards helps to check if it matches with the proper elevation mark on the rod, by appropriate adjustment of the pipe placement.

In the laser method, advantage is taken of an intense, narrow beam of light that is projected by the laser instrument, over a long distance. This beam is aligned through a sewer pipe to strike a target held at the other end of the pipe.

A transit that is placed above a manhole helps establish the alignment of the sewer with reference to field survey points, and transfer it down to the laser instrument that is mounted inside the manhole. Lasers can achieve an accuracy up to 0.01 per cent over a distance of up to 300 m.

## 3.52   CROSS DRAINAGE WORKS

Cross drainage, works arise when a sewer has to cross another service like electricity, water line, gas piping, telecommunication cable, river course, nalah, etc. The following shall be mandatorily implemented without fail.

In regard to the electric power cables, the sewer shall be laid above the electric power cable and horizontally away from the power cable with clearances of minimum 30 cm all round as per IS: 1255. In regard to water lines, the sewer shall always travel below the water line.

With regard to gas lines, the sewer has to travel above the gas line so that sewer gases, if they escape, need not accidentally set off an ignition of the gas line. With regard to telecommunication cables, lateral separation of at least 30 cm shall be followed. In cases of river crossing and nalah crossing, each situation shall be decided on its site conditions.

Gravity sewers, if possible, may be converted to pumped sewer lines by a low lift dedicated pumping station, before the crossing discharging into the gravity section after crossing the water course; this will help in keeping the pumped sewer visible to the eye or close to the ground at all times.

## 3.53   SEWER VENTILATORS

In a modern, well-designed sewerage system, there is no need to provide ventilation on such elaborate scale considered necessary in the past, especially with the present day policy to omit intercepting traps in house connections.

The ventilating columns are not necessary where intercepting traps are not provided. It is necessary however, to make provision for the escape of air to take care of the exigencies of full flow and to keep the sewage as fresh as possible, especially in outfall sewers. In case of storm sewers, this can be done by providing ventilating manhole covers.

## 3.54 PREVENTION OF CROSS CONNECTION

### 3.54.1 Visual Separation

A cross connection between water main and sewer main seldom occurs because of the sizes of these mains. However, where the location is complicated, the water mains shall be either blue coloured pipes or shall be painted with blue florescent coloured paint.

### 3.54.2 Protection of Water Mains

A minimum offset of equal to half the width of the manhole plus 30 cm shall be the lateral offset between water mains and sewer lines. It is advisable to encase the sewer than the water mains.

### 3.54.3 Relation to Waterworks Structures

Gravity sewers shall not be laid closer to water retaining structures and the effort should be to detour as far as possible. In case of leakages in sewer joints, the leakage may gain access to the sidewalls of the water retaining structures.

A simpler precaution if possible will be to use CI or DI pipes for that length of sewer that runs close to the water retaining structure

### 3.54.4 Construction Methods

The design and the construction of sewers are interdependent; the knowledge of one is an essential prerequisite to the competent performance of the other.

The ingenuity of the designer and supervising engineer is continually called for, to reduce the construction cost and to achieve quality workmanship. Barring unforeseen conditions, it shall be the responsibility of the supervising engineer and the contractor to complete the work as shown on the plans at minimum cost and with minimum disturbance of adjacent facilities and structures.

### 3.54.5 Trench

### 3.54.5.1 Dimensions

The width of trench should be the minimum necessary for the proper installation of the sewer with the due consideration to its bedding. It depends on the type of shoring (single stage or two stage), working space required in the lower part of the trench and the type of ground below the surface. The width of the trench at different levels from the top of the sewer to the ground surface is primarily related to its effect upon the adjoining services and nearby structures.

In undeveloped areas or open country, excavation with side slope shall be permissible from the top of the sewer to the ground surface instead of vertical excavation with proper shoring. In developed areas, however, it is essential to restrict the trench width to protect the existing facilities and properties and to reduce the cost of restoring the surface. Increase in width over the minimum required would unduly increase the load on the pipe.

### 3.54.5.2    Excavation

Excavation for sewer trenches for laying sewers shall be in straight lines and to the correct depths and gradients required for the pipes as specified in the drawings. The material excavated from the trench shall not be deposited very close to the trench to prevent the weight of the materials from causing the sides of the trench to slip or fail. The sides of the trench shall, however, be supported by shoring where necessary to ensure proper and speedy excavation. In case, the width of the road or lane where the work of excavation is to be carried out is so narrow as to warrant the stacking of materials near the trench, the same shall be taken away to a place to be decided by the Engineer-in-Charge. This excavated material shall be brought back to the site of work for filling the trench. In case the presence of water is likely to create unstable soil conditions, a well point system erected on both sides of the trench shall be employed to drain the immediate area of the sewer trench prior to excavation operation. A well point system consists of a series of perforated pipes driven into the water bearing strata on both sides of a sewer trench and connected with a header pipe and vacuum pump. If excavation is deeper than necessary, the same shall be fitted and stabilized before laying the sewer.

### 3.54.5.3    Shoring

The shoring shall be adequate to prevent caving in of the trench walls by subsidence of soil adjacent to the trench. In narrow trenches of limited depth, a simple form of shoring shall consist of a pair of 40 to 50 mm thick and 30 cm wide planks set vertically at intervals and firmly fixed with struts. For wider and deeper trenches, a system of wall plates (Wales) and struts of heavy timber section is commonly used. Continuous sheeting shall be provided outside the wall plates to maintain the stability of the trench walls. The number and the size of the wall plates shall be fixed considering the depth of trench and type of soil. The cross struts shall be fixed in a manner to maintain pressure against the wall plates, which in turn shall be kept pressed against the timber sheeting by means of timber wedges or dog spikes. In non-cohesive soils combined with considerable ground water, it may be necessary to use continuous interlocking steel sheet piling to prevent excessive soil movements by ground water percolation and extend the piling at least 1.5 m below the trench bed. In case of deep trenches, excavation and shoring may be done in stages.

A mechanized shoring is presented in Figure. 3.49



Figure 3.49
Work in execution by latest technology for sewer laying in India (Mumbai). Steel anchors have grooves for slotting precast RCC slabs or other sheeting and the anchors are held in place by steel adjustable struts. Specific advantage is easiness of pulling out the anchors, sheeting and struts by deploying mechanical equipment.
Source: Bihar Urban Development Corporation Detailed Project Report for Saidpur Sewer Network - Patna (Package 6)

### 3.54.5.4    Underground Services

All other services like pipes, ducts, cables, mains and other services exposed due to the excavation shall be effectively supported.

### 3.54.5.5    Dewatering

Trenches for sewer construction shall be dewatered for the placement of concrete and laying of pipe sewer or construction of concrete or brick sewer and kept dewatered until the concrete foundations, pipe joints or brick work or concrete have cured. The pumped-out water from the trenches shall be disposed off in existing storm water drainage arrangement nearby.

In the absence of any such arrangement, the pumped water may be drained through completed portion of sewer to a permanent place of disposal. Where a trench is to be retained dry for a sufficient period to facilitate the placement of forms for sewer construction, an under drain shall be laid of granular material leading to a sump for further disposal. Precautions are to be taken to arrest potential floating of the laid sewers, arising out of induced buoyancy during rainy season.

### 3.54.5.6    Foundation and Bedding

Where a sewer has to be laid in a soft underground stratum 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 sewer should be laid on suitable supports or concrete cradle supported on piles.

In the case of cast-in-situ sewers, 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.

### 3.54.5.7    Tunnelling

Tunnels are employed in sewer systems when it becomes economical, considering the nature of soil to be excavated and surface conditions with reference to the depth at which the sewer is to be laid. Generally, in soft soils the minimum depth is about 10 m. In rocks, however, tunnels may be adopted at lesser depths. In busy and high activity zones, crowded condition of the surface, expensive pavements or presence of other service facilities near the surface sometimes, make it advantageous to tunnel at shallower depths. Each situation has to be analysed in detail before any decision to tunnel is taken.

### 3.54.5.8    Shafts

Shafts are essential in tunneling to gain access to the depth at which tunnelling is to be done to remove the excavated material. The size of shaft depends on the type and size of machinery employed for tunneling, irrespective of the size of the sewer.

## 3.55    METHODS OF TUNNELLING

The tunneling methods adopted for sewer construction can be classified generally as auger or boring, jacking and mining.

### a)    Auger or Boring

In this method, rigid steel or concrete pipes are pushed into the ground to reasonable distances and the earth is removed by mechanical means from the shaft or pit location. Presence of boulders is a serious deterrent for adoption of this method, in which case it may be more economical to first install an oversize lining by conventional tunnelling or jacking and fill the space between the pipe and lining with sand, cement or concrete.

### b)    Jacking

In this procedure, the leading pipe is provided with a cutter or edge to protect the pipe while jacking. Soil is gradually excavated and removed through the pipe as successive lengths of pipes are added between the leading pipe and the jacks and pushed forward taking care to limit the jacking up to the point of excavation. This method usually results in minimum disturbance of the natural soils adjacent to the pipe. The jacking operation should continue without interruption as otherwise soil friction might increase, making the operation more difficult.Jacking of permanent tunnel lining is generally adopted for sewers of sizes varying from 750 to 2,750 mm, depending upon the conditions of soil and the location of the line.

The pipes selected should be able to withstand the loads exerted by the jacking procedure. The most common pipes used for this are reinforced concrete or steel.

### c)    Mining

Tunnels larger than 1.5 m are normally built with the use of tunnel shields, boring machines or by open face mining depending on the type of material met with. Rock tunnels normally are excavated open-face with conventional mining methods or with boring tools. These are used as a safety precaution in mining operations in very soft clay or in running sand especially in built up areas. In this method, a primary lining of adequate strength to support the surrounding earth is installed to provide progressive backstop for the jacks which advance the shield.

As the excavation continues, the lining may be installed either against the earth, filling the annular space by grouting with pea gravel or the lining may be expanded against the earth, the latter eliminating the need for any grouting. Boring machines of different types have been developed for tunnel excavation in clay and rock and are equipped with cutters mounted on a rotating head, which is moved forward.

The excavated earth is usually carried by a conveyor system. Some machines are also equipped with shields. Though the machines are useful in fairly long runs through similar material, difficulties are encountered when the material varies. Open face mining without shields are adopted in particular instances such as in rock. Segmental support of timber or steel is used for the sides and the top of the tunnel.

## 3.56 LAYING OF PIPE SEWERS

In laying sewers, the centre of each manhole shall be marked by a peg. Two wooden posts 100 mm x 100 mm and 1,800 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 and shall be fixed at distances more than 30 m apart along the sewer alignment. The centre line of the sewer 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 sewers 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 metres 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 diameter 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 not be removed in any case until the trench is excavated, the pipes are laid, jointed and the filling is started.

When large sewer 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 sewer line, as rough grade for the sewer is completed, would serve the purpose.

### 3.56.1 Stoneware Pipes

The stoneware pipes shall be laid with sockets facing up the gradient, on desired bedding. Special bedding, hunching or encasing may be provided where conditions so demand (as discussed in Section 3.50). All the pipes shall be laid perfectly true, both to line and gradient, IS 4127. At the close of each day's work or at such other times when pipe is not being laid, the end of the pipe should be protected by a close fitting stopper.

### 3.56.2 RCC Pipes

The RCC pipes shall be laid in position over proper bedding, the type of which may be determined in advance, 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 "O" ring by appliances to compress roughly the "O" 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 RCC pipes shall be laid in a manner similar to stoneware spigot and socket pipes. The structural requirements as discussed in this chapter and IS 783 may be followed.

### 3.56.3    Cast-in-situ Concrete Sections

For sewer 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.

### 3.56.4    Construction of Brick Sewers

Sewers larger than 2 m are generally constructed in brick work. The brickwork shall be in cement mortar of 1:3 and plastered smooth with cement plaster of 1:2, 20 mm thick both from inside and outside. A change in the alignment of brick sewer shall be on a suitable curve conforming to the surface alignment of the road. The construction shall conform to IS 2212 in general.

### 3.56.5    Cast Iron Pipes

The pipes shall be laid in position with the socket ends of all pipes facing up gradient. When using lead joints, any deviations either in plan or in elevation of less than 11 1/4 degree shall be effected by laying the straight pipes round the flat curve of such radius that the minimum thickness of lead in a lead joint at the face of the socket shall not be reduced below 6 mm. The spigot shall be carefully pushed into the socket with one or more laps of spun yarn wound round it. Each joint shall be tested before running the lead, by passing completely round it, a wooden gauge notched out to the correct depth of lead and the notch being held close up against the face of socket. When using the "O" ring joints, each "O" ring shall be inserted fully and verified by a toll with prior marking of the socket depth, which, when inserted after the "O" ring joint will reveal that the "O" ring has been fully inserted in position. Special precautions by manufacturers, if any, shall also be followed. Flange joints shall be used with appropriate specials and tail-pieces when inserting a fitting like a meter or a valve in the pipeline. IS 3114 should be followed in setting out the sewers.

### 3.56.6    Ductile Iron Pipes

The same procedures and precautions for laying as in the case of cast iron pipes shall apply here also.

### 3.56.7    Solid Wall UPVC Pipes

The single most important precaution is to ensure that the excavated trench is not water logged. Where situations imply water logging, it is mandatory to employ a well point dewatering system running 24 hours, 7 days a week to hold the subsoil water at least 50 cm below the bedding elevation. Thereafter, the grade of the trench having been checked, lower the pipe with socket ends facing the up gradient. When a pipe needs to be cut to suit a given distance, the pipe shall be cut perpendicular to its axis, using a firm hand held saw. Then bevel the cut end by a bevelling tool or power tool to the same angle as in the original uncut pipe and mark the insertion line freshly using an indelible black paint to retain the guide limit for insertion. Carefully remove any loose soil from the socket and do not remove the "O" ring from its housing. Check by hand whether the "O" ring is seated uniformly. Thereafter, place the pipe spigot end near the socket.

**3.56.8    Solid Wall HDPE Pipes**

Unlike in the case of CI, DI, UPVC pipe sewers, the HDPE sewers are normally butt welded and pre-assembled on ground and only then lowered inside the trench spanning manhole to manhole. The  butt-welding shall follow the manufacturer's recommendations. Where flanged joints are needed for attaching or inserting fittings and specials like valves, the free end of the HDPE pipe shall be butt-welded with a standard flange and thereafter the flanged jointing can be made. However, in the case of such pipes, the uplift during high groundwater conditions above the pipe level is a problem specifically in high ground water and coastal areas. The concrete surrounds or venteak piles shall be used to hold these in place in such conditions, where ground water can rise above the sewer.

**3.56.9    Structured Wall Pipes**

The IS 16098 (Part-1), IS 16098 (Part-2) and EN 13476 also cover the performance requirements for the respective materials. These pipes are manufactured with externally corrugated wall or with T-beam type of wall with hollows between the webs of the T beams. These are laid in almost the same way as the UPVC pipes. These outer-ribbed wall pipes are jointed with "O" rings after due cleaning of dust, etc., using push-tight method and these rings help in preventing the escape of the contained fluid.

**3.56.10    Double Wall Corrugated Polyethylene Pipes**

Please refer section 3.12.9.3.

**3.56.11    Relative Limitations in Pipe Materials in Some Situations**

The merits and demerits of different pipe materials are covered in Section 3.12 and their laying is covered in this Section 3.56.

It will be useful to keep in mind that sewers pass through a whole length of roads in a habitation and varying soil conditions, bedding conditions, locations, etc., will be encountered at various places. Hence, a particular pipe material may be suitable in a particular location but may require some other material at some other locations.

A guide for this is presented in Appendix A.3.10. This may be referred to during field execution of the sewer pipes and necessary local adjustments can be made.

**3.57    LOAD CARRYING MECHANISM OF THE PIPES**

The non-metallic and non-concrete sewer pipes behave integral with the surrounding soil when it comes to structural behaviour. As loads are superimposed, the pipe cross section may tend to deflect by marginal reduction in vertical diameter. This may induce an increase in horizontal diameter,  but this increase will be resisted by the lateral soil pressure and eventually there arises a near uniform radial pressure around the pipe and a compressive thrust. This is illustrated in Figure 3.50 overleaf.
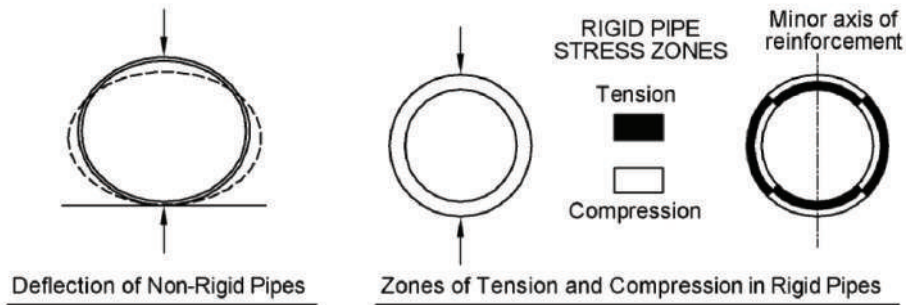
Figure 3.50  Mechanism of resisting superimposed loads by rigid and non-rigid piping

Thus, adequate backfilling in layers and compaction in each layer is of paramount importance. On the contrary, the metallic and concrete pipes, being stiffer than the surrounding soil, they carry a substantial portion of the applied load but the shear stress in the haunch area can be critical specifically,  when haunch support is inadequate. Thus the load carrying mechanism of both these classes of pipe materials are dependent on the haunch supports and proper backfilling.

## 3.58    JOINTING OF SEWERS

Joints of pipe sewers may generally be any of the following types:

i)    Spigot and socket joint (rigid and semi flexible)

ii)   Collar Joint (rigid and semi flexible)

iii)  Cast Iron detachable joint (semi flexible)

iv)  Coupling joint (semi flexible)

Cement joints are rigid and even a slight settlement of pipes can cause cracks and hence leakage. To avoid this problem it is recommended that semi flexible joints be used.

### 3.58.1    Stoneware Pipes

All the pipe joints shall be 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 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. A fillet shall be formed round the pint with a trowel forming an angle of 45 degrees with the barrel of the pipe IS 4127. Rubber gaskets may also be used for jointing. A method of relatively easier checking of the grade of SW pipe sewer line is followed by the CMWSSB. In this method, two tight strings connected to the crown and one horizontal diameter edge as shown in Figure 3-51 (overleaf) are used to judge and adjust the grade, which is much faster and more precise than the boning rod method which becomes cumbersome.

### 3.58.2    Concrete Pipes

Concrete spigot and socket pipes are laid and jointed as described above for glazed stoneware spigot and socket pipes with yarn or rubber gasket and cement.

Source: CMWSSB, Chennai, 2012

Figure 3.51  A simpler approach to initial laying and aligning the SW sewers by the two tight twines one on crown and one on mid diameter are used

Asbestos cement pipes are joined by coupling joints or CI detachable joints.

Large size concrete sewers have 'ogee' joints in which the pipe has mortise at one end and a tendon to suit at the other end. They are jointed with cement or asphalt. A concrete collar sufficiently wide to cover and overlap the joint is fixed on it.The collars shall be placed symmetrcally over the end of two pipes and the annular space between the inside of the collar and the outside of the pipe shall be filled with hemp yarn soaked in tar or cement slurry tamped with just sufficient quantity of water to have a consistency of semi dry condition. This is well packed and thoroughly rammed with caulking tools and then filled with cement mortar (1:2) prop. The joints shall be finished off with a fillet sloping at 45 degrees to the surface of the pipe. The finished joints shall be protected and cured for at least 24 hours. Any plastic solution or cement mortar that may have squeezed in shall be removed to leave the inside of the pipe perfectly clean. For more details of jointing procedure, reference may be made to IS 783.

### 3.58.3    Cast Iron Pipes

For CI pipes several types of joints such as rubber gasket known as Tyton joint, mechanical joint known as screw gland joint and conventional joint known as lead joint are used. For details refer to CPHEEO Manual on Water Supply and Treatment and relevant Indian Standards.

### 3.58.4    Ductile Iron pipes

The same procedures and precautions as in the case of cast iron pipes shall apply here too.

### 3.58.5    Solid Wall UPVC Pipes

Just before jointing, the lubricating material supplied by the pipe manufacturer shall be uniformly applied around the spigot end and onto the "O" ring surface to be in contact with  the spigot end after jointing.

Do not remove the O ring for doing this. Thereafter use the lateral force by pushing the socket end of the pipe to be inserted by placing a wooden plank across its face and using a crowbar plunged and anchored into the soil as a level. Do not try to hit the pipe socket. When the insertion mark is reached at the face of socket, stop the work.

### 3.58.6   Solid Wall HDPE Pipes

The jointing is by welded heat fusion of the pipe cut surfaces to be jointed. The temperature and the time of contact are generally specified by the manufacturer.

### 3.58.7   Structured Wall Pipes

These are mainly by "O" ring gaskets inserted in a spigot-socket arrangement. In the case of T-Beam wall type pipes, these ends are made integral with the pipe. In the case of externally corrugated pipes, once the pipe ends are positioned and verified for alignment, lubricate the "O" ring in the correct slit as indicated by the manufacturer and push the coupling to its designated location. The "O" ring is fixed into these circular recesses and the piping is slid over it using a separate coupling, which slides over the "O" ring and brings about the jointing. Structured wall pipes are laid and jointed between them or between structured wall and solid wall type cells. The precautions will be to make sure that pipe ends and couplings are cleaned free of extraneous matter and test slide the coupling to mark the pipe ends at half the coupler length to ascertain the lengths of the pipe inside the coupling.

### 3.58.8   Double Walled Corrugated Pipes

These are jointed the same way as externally corrugated structured wall piping.

### 3.59   PRECAUTIONS AGAINST UPLIFT

Other than the metallic and concrete pipe sewers, the uplift during high groundwater conditions above the pipe level is a problem specifically in high ground water locations, water logged locations and coastal areas. The concrete surrounds or venteak piles shall be used to hold these in place in such conditions, where the ground water can rise above the sewer.

### 3.60   THE WATER JETTING ISSUES

With the Honourable courts levying huge penalties if man entry is practiced for sewer cleaning and with the commitment of the ULB to do away with this practice, the mechanical methods of sewer cleaning have gained momentum and is being practiced more widely in recent years. These machines, which are popularly known as jet rodders, jet the water or secondary treated sewage into the sewers by a jack-hammer action at high pressures as in Figure 3.10.

The ability of the stoneware, cast iron, concrete, HDPE/PE/PP/PVC sewer pipes to withstand the pressures to establish the permissible pressure ratings has to be evolved in India. However, an available literature is from the Sewer Jetting Code of Practice first published in 2001 in UK and which provides guidance on jetting pressure for different types of sewer pipes, as in Table 3-27 overleaf.

Table 3.27  Maximum jetting pressure in case of different types of pipes

| No. | Maximum Jetting Pressure | Concrete | Clay | Plastic | Bricks/fibre |
|-----|--------------------------|----------|------|---------|--------------|
| 1 | Meter of Water | 3450 | 3450 | 1800 | 1030 |
| 2 | BAR | 345 | 345 | 180 | 103 |

Source: Water Research Centre, 2005

Care must be exercised in the field when applying the pressures to clean the sewers since the pressures stated in Table 3.27 are the maximum pressures.

## 3.61    TESTING OF SEWER LINES

### 3.61.1    Water Test

Each section of sewer shall be tested for water tightness preferably between manholes. To prevent change in alignment and disturbance after the pipes have been laid, it is desirable to backfill the pipes up to the top, keeping at least 90 cm length of the pipe open at the joints. However, this may not be feasible in the case of pipes of shorter length, such as stoneware and RCC pipes. With concrete encasement or concrete grade, partial covering of the pipe is not necessary.

In case of concrete and stoneware pipes with cement mortar joints, pipes shall be tested three days after the cement mortar joints have been made. It is necessary that the pipelines be filled with water for about a week before commencing the application of pressure to allow for the absorption by pipe wall. The sewers are tested by plugging the ends with a provision for an air outlet pipe with stop-cock in the upper end. The water is filled through a funnel connected at the lower end provided with a plug. After the air has been expelled through the air outlet, the stop-cock is closed and water level in the funnel is raised to 2.5 m above the invert at the upper end. Water level in the funnel is noted after 30 minutes and the quantity of water required to restore the original water level in the funnel is determined. The pipe line under pressure is then inspected while the funnel is still in position. There shall not be any leaks in the pipe or the joints (small sweating on the pipe surface is permitted). Any sewer or part thereof not meeting the test shall be emptied and repaired or re-laid as required and tested again.

The leakage or quantity of water to be supplied to maintain the test pressure during the period of 10 minutes shall not exceed 0.2 litres/mm dia. of pipes per kilometre length per day.

For non-pressure pipes, it is better to observe the leakage for a period of 24 hours if feasible. The test for exfiltration for detection of leakage shall be carried out at a time when the groundwater table is low.

For concrete, RCC and asbestos cement pipes of more than 800 mm dia. the quantity of water inflow can be increased by 10% for each additional 100 mm of pipe dia.

For brick sewers, regardless of their diameters, the permissible leakage of water shall not exceed 10 cubic meters for 24 hours per km length of sewer.

### 3.61.2   Air Testing

Air testing becomes necessary particularly in large diameter pipes when the required quantity of water is not available for testing. As per the ASTM C28-80, vitrified clay pipes testing is specified as applying air pressure to 2.8 m water column and held for 2 to 5 minutes when all plugs are checked and the exact point of leakage can be detected by applying soap solution to all the joints in the line and looking for air bubbles. Thereafter, the air supply is disconnected and the time taken to drop from 2.5 m to 1.7 m water column for every 30 m is noted to be in conformity with Table 3.28.

Table 3.28  Minimum test times per 30 m of vitrified clay sewer line for air testing

| Diameter, mm | minutes | Diameter, mm | minutes | Diameter, mm | minutes |
|---|---|---|---|---|---|
| 100 | 0.3 | 400 | 2.1 | 750 | 4.8 |
| 150 | 0.7 | 450 | 2.4 | 800 | 5.4 |
| 200 | 1.2 | 500 | 3.0 | 900 | 6.0 |
| 250 | 1.5 | 600 | 3.6 | 950 | 6.6 |
| 300 | 1.8 | 700 | 4.2 | 1,070 | 7.3 |

The longer lengths and hence fewer joints of sewer pipelines when laid with RCC and double walled HDPE pipes must be able to easily withstand the above testing and hence, the same test conditions are retained for these sewers also. A typical arrangement is shown in Figure 3.52.
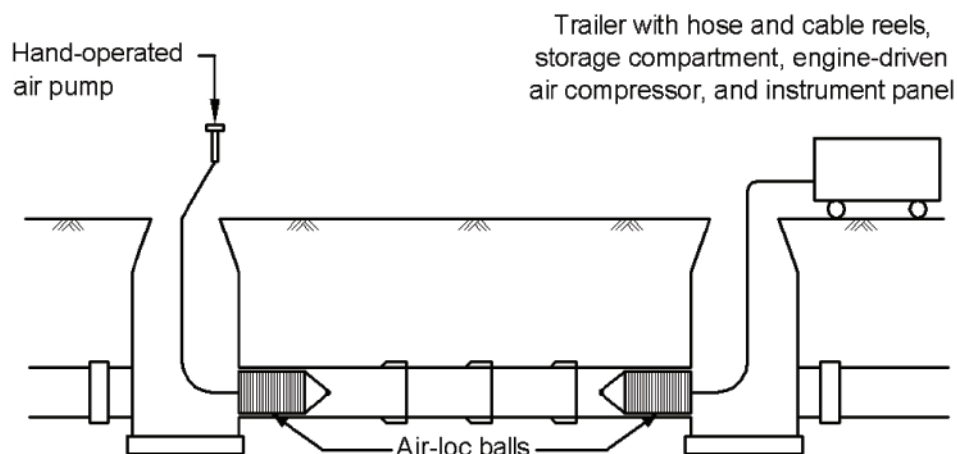


Figure 3.52  Typical arrangement for low pressure air testing of sewer pipeline

### 3.62   CHECK FOR OBSTRUCTION

As soon as a stretch of sewer is laid and tested, a double disc or solid or closed cylinder, 75 mm less in dimension than the internal dimension of the sewer shall be run through the stretch of the sewer to ensure that it is free from any obstruction.

## 3.63    BACKFILLING OF THE TRENCHES

Backfilling of the sewer trench is a very important consideration in sewer 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 load 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.

No trench shall be filled in unless the sewer stretches have been tested and approved for water tightness of the joints. However, partial filling may be done keeping the joints open to avoid any disturbance. The refilling shall proceed around and above the pipes. Soft material screened free from stones or hard substances shall first be used and hand pressed under and around the pipes to half their height. Similar soft material shall then be put up to a height of 30 cm above the top of the pipe and this will be moistened with water and well rammed. The remainder of the trench can be filled with hard material, in stages, each not exceeding 60 cm. At each stage, the filling shall be well rammed, consolidated and completely saturated with water and then only further filling shall be continued. Before and during the backfilling of a trench, precautions shall be taken against the floatation of the pipeline due to entry of large quantities of water into the trench causing an uplift of the empty or the partly filled pipeline. Reference may be made to section 3.46 for more details in this regard. Upon completion of the backfill, the surface shall be restored fully to the level that existed prior to the construction of the sewer.

## 3.64    REMOVAL OF SHEETING

Sheeting driven below the spring line of a sewer shall be withdrawn a little at a time as the back-filling progresses. Some of the backfilled earth is forced into the void created by withdrawing the sheeting by means of a water jet. To avoid any damage to buildings, cables, gas mains, water mains, sewers, etc., near the excavation, or to avoid disturbance to the sewer already laid portions of the sheeting may be left in the trenches.

## 3.65    SEWER REHABILITATION

Disrepair of sewers renders them leaky; and, as a result, they carry large volumes of infiltration water. They, most often get, blocked and sometimes collapse. The expenditure of excavating and then replacing a portion of badly functioning sewer is prohibitive. It is, therefore, economical to repair and rehabilitate the system as such. Therefore, continuing sewer maintenance efforts have to be designed with a view to preventing unnecessary deterioration of the sewer system. Any maintenance programme that may be adopted depends on the nature of the problem, necessity of maintaining the flow while the repair is being carried out, the expected traffic disruption that may be caused, safety aspects that need be addressed, and the cost that has to be borne.

It is necessary to clean the sewer lines before embarking on a visual inspection. This is commonly done by flushing the sewer by using a fire hose, connected to a hydrant, which discharges into a manhole.

However, caution is to be applied to avoid backups into the surrounding buildings that are connected to the system. Another method to clean the sewers is by using a soft rubber ball that is inflated to match the diameter of the pipe and later being pulled by a cord via the reach of the line between manholes. Power rodding machines or power winches (to pull a bucket through the line) can also be used. Moreover, it is to be looked into that the collected debris is disposed of properly. Inspections, after cleansing operations, are made during low-flow periods using flashlights. Use of closed-circuit television system (even making a photographic or videotape record) gives accurate location of leaks, root intrusions and any structural problems. A common method for sealing leaks in otherwise structurally sound pipelines comprises chemical grouting, the grout is applied internally to joints, holes, and cracks. In smaller or medium sized lines, inflatable rubber sleeves are generally pulled through, while in large sized lines workers place a sealing ring manually over the defective joint and, the grout is pumped through a hand held probe. However, as a safety measure, the air in the sewer must be tested for carbon monoxide, hydrogen sulphide, and explosive gases before allowing entry to workers.

Crown corrosion can cause structural damage to sewers. Large sewers, suffering this damage can be strengthened by applying a lining of gunite, a mixture of fine sand, cement, and water. It is applied internally by means of pneumatic spraying. Quite long lengths of concrete sewers are effectively rehabilitated with gunite lining. To renew on extensively cracked sewer lines a procedure known as slip lining is adopted. It comprises pulling a flexible plastic liner pipe into the damaged pipe and then reconnecting all the individual service connections to the liner. It may be necessary sometimes, to fill the narrow annular space between the lines and the existing pipe with grout preventing relative movement. However, it may be pointed out that multiple excavations are required to reconnect each service line to the new liner. In a relatively new and sophisticated method, namely, Inversion lining, a flexible liner is used. This line expanding to fit over the pipe geometry is thermally hardened and  the procedure avoids excavations for service line connections.

Concrete manholes may also suffer sulphuric acid corrosion. Severe cases may need total replacement of the manhole. For less severe cases, the deteriorated material is removed using water or sand blasting, or mechanical tools, and then special chemical preparations are applied to stabilize the remaining material. Next, high strength patching mortar is used in filling in the irregularities in the internal surface; and lastly a lining or a coating has to be applied.

Manholes are sometimes subject to surface water inflow and/or ground water infiltration, and it is an unacceptable situation. This circumstance can arise due to holes in the manhole cover; spaces between the cover and the frame; and poor sealing of the frame of the cover. Frames can be resealed using hydraulic cement, and water-proof epoxy coating. Sometimes, the manhole frame and cover are raised, and the exposed portion is coated with asphalt or cement. One more method consists in installing a special insert between the frame and the cover and it does not allow water and grit to enter the manhole while allowing gas to escape through a relief valve.

Infiltration of groundwater through the sidewall of a manhole and its base, or around pipe entrances is solved by chemical grouting; being a less costly method compared to lining or coating, it also needs no preparatory restoration of the surface. Further, cracks and opening get sealed by pressure injection of the gel or foam (grouting materials).

House (service) connections and smaller diameter pipes, join the lateral sewer line in the street with the building that the sewer line serves. These house lines are also known as building sewers or service laterals, and can be as long as 30 m. These can develop defects like cracks and open-jointed pipes, causing considerable infiltration of groundwater. The total length of service connections can often be greater than the length of the main sewers. Therefore, the maintenance of these lines is also equally important. Chemical grouting and inversion lining procedures are often helpful.

Sewers, which are determined to be critical after inspection, have to be taken up for rehabilitation. Sewer rehabilitation is necessitated either to improve the hydraulic performance of the existing line or due to danger of the sewer line deteriorating further and leading to eventual collapse or failure.

### 3.65.1   Methods

Sewer rehabilitation may be carried out by renovation or by renewal of the sewer. When the condition of the sewer is improved either to increase its carrying capacity or to increase its life, it is known as renovation. When the sewer line is reconstructed or replaced to the same dimensions as existing, it is known as renewal.

### 3.65.2   Sewer Renovation

While preparing the DPRs for a habitation where a sewerage system is already in place, it is equally important to consider and provide for renovation of old sewers as well especially, when the old and augmented systems will be functioning contingent upon each other. In the renovation of sewers, the original sewer fabric is utilized and improvements are carried out; the various methods utilized are:

a)   Stabilization where painting or chemical grouting of the joints is carried out

b)   Pipe linings in which pipes of slightly smaller diameter than the sewer are inserted

Pipes may be of Glass Reinforced Plastic (GRP) and HDPE which can be butt fusion welded. The in-situ tube, manufactured of polyester felt and impregnated with a resin mixed with a special catalyst is tailored to suit the internal diameter/dimension of the pipe. The in-situ tube is inserted from any manhole, opening, etc. During insertion, the tube turns inside out so that the polyurethane side forms the inside surface of the pipe. Water is pumped into the tube to a predetermined head and the tube travels down the pipe to be repaired. As the in-situ tube travels, the pressure of the water firmly presses the resin impregnated side against the pipe wall. When the in-situ tube reaches the downstream manhole, addition of water is stopped and the water heated to cure the resin. The result is a cast in-situ pipe within a pipe. An alternative method of pulling the tube in and then inflating it is also used for small diameter pipes. Recent development is the use of photo curing resins, i.e., curing by light.

c) Segmental linings of glass reinforced cement, GRP, resin concrete and precast gunite are used when man entry is possible

d) When linings are used, annulus grouting is necessary in majority of the cases for a satisfactory performance.

e) The places where sewer network is crossing a canal/distributary or a natural drain, the site conditions need to be assessed and analyzed carefully considering various available options. At the locations where sewer network is crossing a natural drain, the depth of sewer is kept in such a way that it crosses below the bed level and can be laid through open trenches. The places where large diameter network pipes are crossing a canal or distributary the crown of the pipe is kept more than "D" m below the bed level, where "D" is the diameter of the pipe so as to ensure the pipes can be laid using trenchless technology without disturbing the canal above. The criteria for selection of trenchless technology based on the diameter of pipe is presented in Table 3.29.

Table 3.29  Criteria for selection of trenchless technology

| No. | Diameter of Pipe | Suitable Trenchless Technology |
|-----|------------------|-------------------------------|
| 1 | < 1,000 mm | Guided Boring |
| 2 | 1,000 mm – 1,500 mm | Pipe Jacking |
| 3 | > 1,500 mm | Tunnel Boring |

f) Different trenchless technologies are used for different diameters, material of construction of pipe and site conditions. These are explained in the subsequent sections.
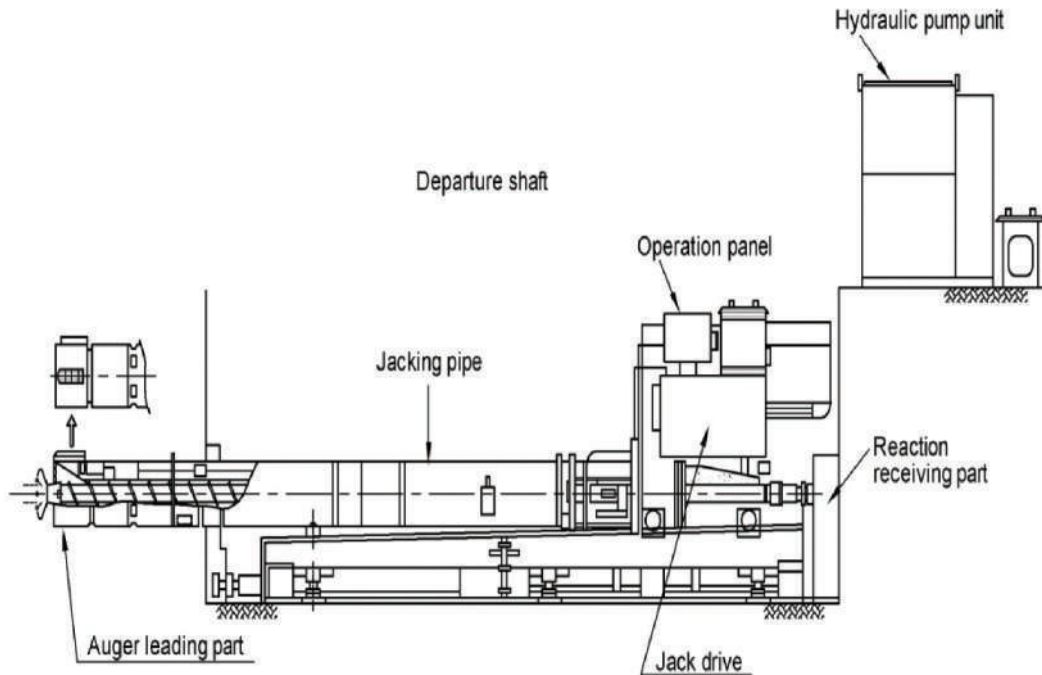
g)  Guided Auger Boring

Auger boring is a technique for forming a horizontal bore under a crossing, using a cutting head and auger flights. The auger boring equipment consists of a cutting head attached to a helically wound auger flight. The rotation action of the auger flight simultaneously rotates the cutting head and removes the excavated soil from the bore. The auger flight is contained within a steel casing.

In auger boring, the auger rotates inside the casing as it is jacked. Hence, there is a danger that it may damage any interior coating or liner that may be in the pipe. The standard casing material used with auger boring is steel. Presently, most of the rail road and highway specifications require the use of steel casing with auger boring. The cutting head and auger are rotated from the drive pit by a transmission or power unit. Most auger boring systems include pipe-jacking equipment, which allows the casing to be moved forward as the cutting head advances. Once the casing has been installed,  the product pipe can be inserted. The size of pipe that can be installed by this method ranges from 100 mm to more than 1,500 mm. However, the most common size range is 200 mm to 900 mm. A typical guided auger boring is shown in Figure 3.53 overleaf.

h)  Pipe Jacking

Pipe jacking is a trenchless construction method, which requires workers inside the jacking pipe and is generally started from an entry pit and can be done manually or by using machines. However, it is accomplished with workers inside the pipe. The excavation method varies from the very basic process of workers digging the face with pick and shovel to the use of highly sophisticated tunnel boring machines.

Source:JSWA, 1991

Figure 3.53  Guided auger boring

Since the method requires personnel working inside the pipe, the method is limited to personnel-entry-size pipes. Hence, the minimum pipe diameter recommended by this method is 1060 mm outside diameter.

Irrespective of the method, the excavation is generally accomplished inside an artificial shield, which is designed to provide a safe working environment for the people working inside and to allow the bore to remain open for the pipe to be jacked in place. The shield is guidable to some extent with individually controlled hydraulic jacks.

The first step in any pipe jacking operation is site selection and equipment selection as per the site requirements. A pipe-jacking project should be planned properly for a smooth operation. The site must provide space for storage and handling of pipes, hoisting equipment for the pipe, spoil storage and handling facility, etc. If adequate space is available, a big jacking pit is preferred so that longer pipe segments can be jacked and the total project duration is reduced.

The jacking pit size is a function of pipe diameter, length of pipe segment, shield dimensions, jack size, thrust wall design, pressure rings and guide rail system. The space available at the site governs the selection of all the above components. The jacking pit should be shored and braced unless it is very shallow and high strength clay. It can be shored with timber, steel piling or shaft liner plates. Due to the jacking forces required to push large diameter pipes through the ground, the jacking pit design and construction are critical. The pit embankment supports must be properly designed and constructed. It is critical that pit floor and the thrust reaction structure be designed to withstand the weight of the heavy pipe segment repeatedly placed on it as well as the continuously exerted jacking loads as the operation is being conducted.

Preparation of the floor of pit, i.e., soil, stone or concrete slab will be determined by the length, size and/or duration of the job. The final alignment and grade will depend largely on the initial setup. Therefore, it is advisable to set up a concrete slab foundation for large jobs, which are likely to take a long time. The pit should have space for personnel to walk on both sides of the pipe. It is important that the pit should be dry and continuous dewatering provisions should be made.

One of the major factors that affect pipe jacking is the jacking force required to push the pipe inside the soil. Every effort is made to minimize the thrust. Application of bentonite to the outer skin of the jacking pipe reduces the friction between the jacking pipe and the soil, and reduces the thrust requirements.

The use of intermediate jacking stations (IJS) is common to control or increase the jacking forces. There is no limit to the number of IJS that can be installed in a line. The IJS permit the pipe to be thrust forward in sections rather than the total length being thrust forward from the jacking pit.

The manual pipe-jacking method is suitable for diameter up to 1500 mm. For large diameters manual jacking is not advisable as grade and alignment maintenance may not be possible in such cases. For such works, mechanical techniques like utility tunnelling / Tunnel Boring Machine (TBM) may be chosen. A typical trenchless pipe jacking method is shown in Figure 3.54.
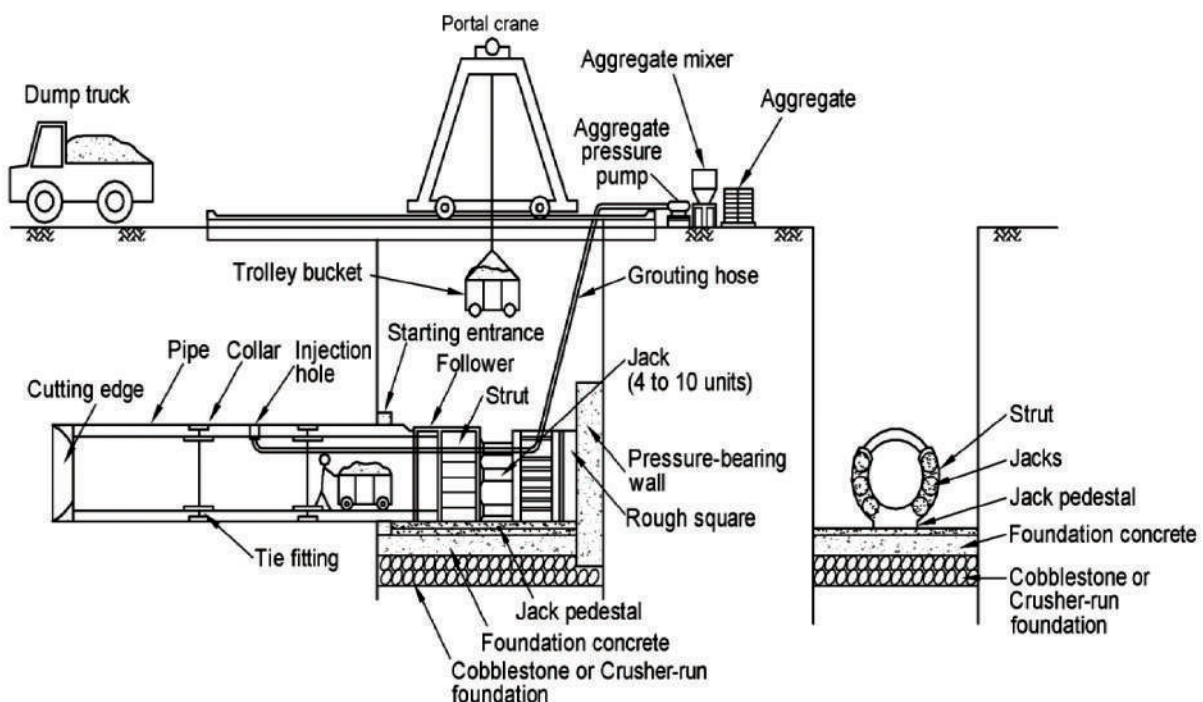


Source:JSWA, 2011

Figure 3.54  Trenchless pipe jacking

i) Tunnel Boring / Utility tunnelling / Trenchless Technology

Tunnel boring is pipe-jacking method; but in this method, instead of manual excavation, highly sophisticated tunnel boring machines are used for excavation. It is generally used for diameter higher than 1,500 mm and where proper slope / gradient is required.

### 3.65.3    Illustrative Example

Illustrative example for structural design of buried sewer is given in Appendix A.3.8.

## 3.66    STORMWATER RELATED STRUCTURES

These are devices meant to transmit the surface runoff to the sewers in the case of combined system and form a very important part of the system. Their location and design should therefore be given careful consideration.

Storm water inlets may be categorized under three major groups viz. curb inlets, gutter inlets and combination inlets, each being either depressed or flush depending upon their elevation with reference to the pavement surface.

The actual structure of an inlet is usually made of brickwork. Normally, cast iron gratings conforming to IS 5961 shall be used. In case there is no vehicular traffic, fabricated steel gratings may be used. The clear opening shall not be more than 25 mm.

The connecting pipe from the street inlet to the main street sewer should not be less than 200 mm in diameter and should have sufficient slope. Maximum spacing of inlets would depend upon various conditions of road surface, size and type of inlet and rainfall. The maximum horizontal spacing of 30 m is recommended.

### 3.66.1    Curb Inlets

Curb inlets are vertical openings in the road curbs through which the storm water flows and are preferred where heavy traffic is anticipated.

They are termed as deflector inlets when equipped with diagonal notches cast into the gutter along the curb opening to form a series of ridges or deflectors. This type of inlet does not interfere with the flow of traffic as the top level of the deflectors lie in the plane of the pavement.

### 3.66.2     Gutter Inlets

These consist of horizontal openings in the gutter, which is covered by one or more gratings through which the flow passes.

### 3.66.3    Combination Inlets

These are composed of a curb and gutter inlet acting as a single unit. Normally, the gutter inlet is placed right in front of the curb inlets but it may be displaced in an overlapping or end-to-end position. Figure 3.55 (overleaf) shows the different types of inlets.

### 3.66.4    Catch Basins

Catch basins are structures meant for the retention of heavy debris in storm water which otherwise would be carried into the sewer system. Their use is not recommended since they are more of a nuisance and a source of mosquito breeding apart from posing substantial maintenance problems.
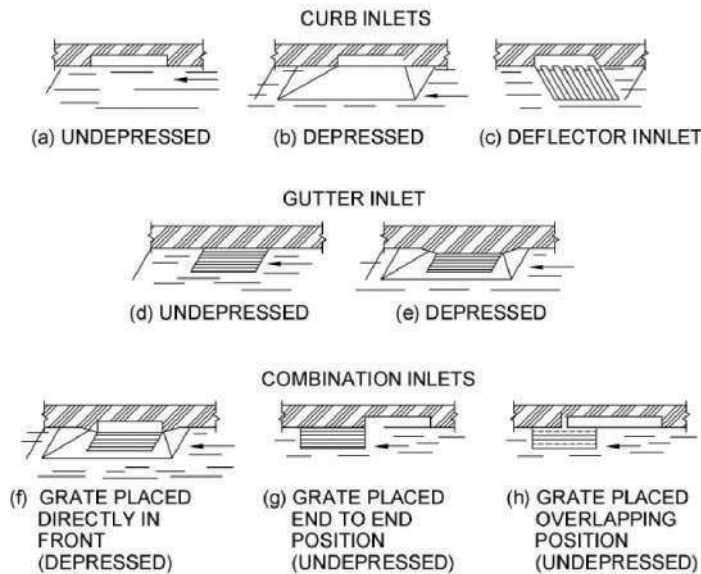
Source: CPHEEO, 1993

Figure 3.55  Different types of inlets

Where a main sewer is laid and the sewer network is not yet laid, the dry weather flow from the open drains may be connected to the sewers by making a provision for a catch basin and overflow weir.

### 3.66.5    Flap Gates and Flood Gates

Flap gates or backwater gates are installed at or near sewer outlets to prevent backflow of water during high tide or at high stages in the receiving stream. Such gates should be designed so that the flap should open at a very small head differential. With a properly operated flap gate, it is possible to continue to pump a quantity equivalent to the sanitary sewage flow from the combined sewer to the treatment plant even though flood conditions prevail in the stream at the sewer outlet.

In case of a sea and estuary outfall, the outfall sewer should be able to discharge at full rate when the water level in the estuary or sea is ¾th the mean annual tide level. Adequate storage to prevent backflow into the system due to the closure of these gates at the time of high tides is also necessary if pumping is to be avoided. To control the flow from the storage tank, flood-gate or penstocks are provided which can be opened and closed quickly at the predetermined states of tide. The gates are generally electrically operated and are controlled by a lunar clock.

Many flap or backwater gates are rectangular and may consist of wooden planks. Circular or rectangular metallic gates are commercially available. Flap gates may be of various metals or alloys as required by the design conditions.

Flap gates are usually hinged by a link-type arrangement that makes it possible for the gate shutter to get seated more firmly. Hinge pins, linkages and links should be of corrosion resistant material. There should be a screen chamber to arrest floating undesirables on the upstream side of the flap gate. The maintenance of flap gates requires regular inspection and removal of debris from the pipe and outlet chamber, lubrication of hinge pins and cleaning of seating surfaces.

## 3.67 OUTFALL SEWERS

The aspects to be considered in the design of a sewer outfall are listed as under:

1) Location to avoid unpleasant sight and offensive smell

2) Protection of the mouth of sewer if it empties into a river against swift currents, water traffic, floating debris, heavy waves, or other hazards which might damage the structure; and

3) Prevention of backing-up of water into the sewer if the outfall is having a flat grade

## 3.68 CROSS INFRASTRUCTURE WORKS

Section 3.52 shall be mandatorily followed.

## 3.69 CORROSION PREVENTION AND CONTROL

### 3.69.1 General

Corrosion is the phenomenon of the interaction of a material with the environment (water, soil or air) resulting in its deterioration. There are many types of corrosion, the major types being galvanic, concentration cell, stray current, stress and bacterial. Sewage collection and treatment systems are more prone to corrosion in view of the nature of the sewage. Since sewage contains solids which are more likely to cause abrasion in sewers, pumps and their components, thus removing the protective coating and accelerating the corrosion process, corrosion control becomes all the more important in sewerage systems. It is particularly acute in areas where sewage strength is high, sulphate content of water is substantial and average temperature is above 20ºC. The corrosion problem in sewerage systems can be categorized as (1) Corrosion of sewers and (2) Corrosion of treatment systems.

### 3.69.2 Corrosion of Sewers

The most widely used materials for sewers are reinforced concrete, stoneware, asbestos cement and cast iron. The development of plastics, fibre glass and other synthetic materials has increased the choice of piping materials. For gravity sewers the usual practice is to use vitrified stoneware pipes for smaller sizes and cement concrete pipes for larger sizes. For pumping mains, CI pipes are generally used. Factors such as climate and topography, high temperature, flat grades and long length of sewers may favour the development of highly septic, sulphide containing sewage in the sewer line. Industrial wastes may aggravate these problems by the introduction of high concentration of pollutants and/or large volumes of hot water that accelerate chemical and biological reaction rates. Concrete sewers are the worst affected because of sulphides in sewage.

### 3.69.3 Corrosion due to Biological Reactions

Hydrogen Sulphide may be produced biologically in sewers by (1) the hydrolysis of organic compounds containing sulphur and (2) by reduction of sulphates. Sewage contains a variety of sulphur bearing organic compounds (usually at concentration between 1 to 5 mg/l) and inorganic sulphates, which find their way through drinking water, industrial water or sea water intrusion.

Hydrogen sulphide in sewer is usually produced by bacteriological reduction of sulphates. Hydrogen sulphide gas by itself is not injurious to cement concrete, unless it gets readily oxidized by dissolved oxygen or by several bacterial species.

Oxygen is normally present in the air between the crown and the sewage, $H_2S$, a prerequisite for sewer corrosion and $CO_2$, are usually present in the sewer air. In the presence of air, $H_2S$ gets oxidized to sulphuric acid and this sulphuric acid reacts with the cement constituents of concrete. In fact, it reacts with the lime in the cement concrete to form calcium sulphate, which in turn, reacts with the calcium aluminates in the cement to form calcium sulpho-aluminates.

Expansion caused by these reactions results in spalling of the surface of the concrete, thereby exposing underlying layers of concrete to further attack. If the corrosion products adhere to the surface of the concrete, then a certain measure of protection against further acid attack is provided. Sulphuric acid, in fact, does not and cannot penetrate into normal concrete.

Acid attack therefore takes place at the surface only. The most outstanding character of this form of corrosion is the fact that it only occurs above the water line in the sewer. In other words, it is the crown portion of the pipe, which gets corroded and this phenomenon is referred to as crown corrosion. Due to this corrosion, the reinforcement gets exposed and the sewer gets damaged. In general, synthetic material pipes are not directly affected by biological corrosion.

### 3.69.4 Factors Influencing Sulphide Generation

The factors that influence sulphide generation in sewers include: (i) temperature of sewage, (ii) strength of sewage, (iii) velocity of flow, (iv) age of sewage, (v) pH of sewage, (vi) sulphate concentration and (vii) ventilation of the sewer.

### 3.69.4.1 Temperature

Since sulphide generation is a biological phenomenon, it is obvious that sewage temperature influences the rate of sulphide generation. Temperature below 20°C generally will not cause any appreciable sulphide build up. From 20°C to 30°C, the rate of sulphide generation increases at about 7% per °C rise in temperature and is maximum at 38°C.

### 3.69.4.2 Strength of Sewage

A high concentration of organic strength (BOD) in sewage will lead to an increased rate of sulphide generation as in Figure 3.56 and Figure 3.57 overleaf.

For any specified sewage temperature and flow condition in a sewer, there is limiting sewage strength, usually less than 80 mg/l of BOD, below which a build-up of hydrogen sulphide will practically cease.

However, it is possible in a long force main or at other locations where oxygen is shut off from the sewage for a few hours, that sulphide build up may occur even with low values of BOD.
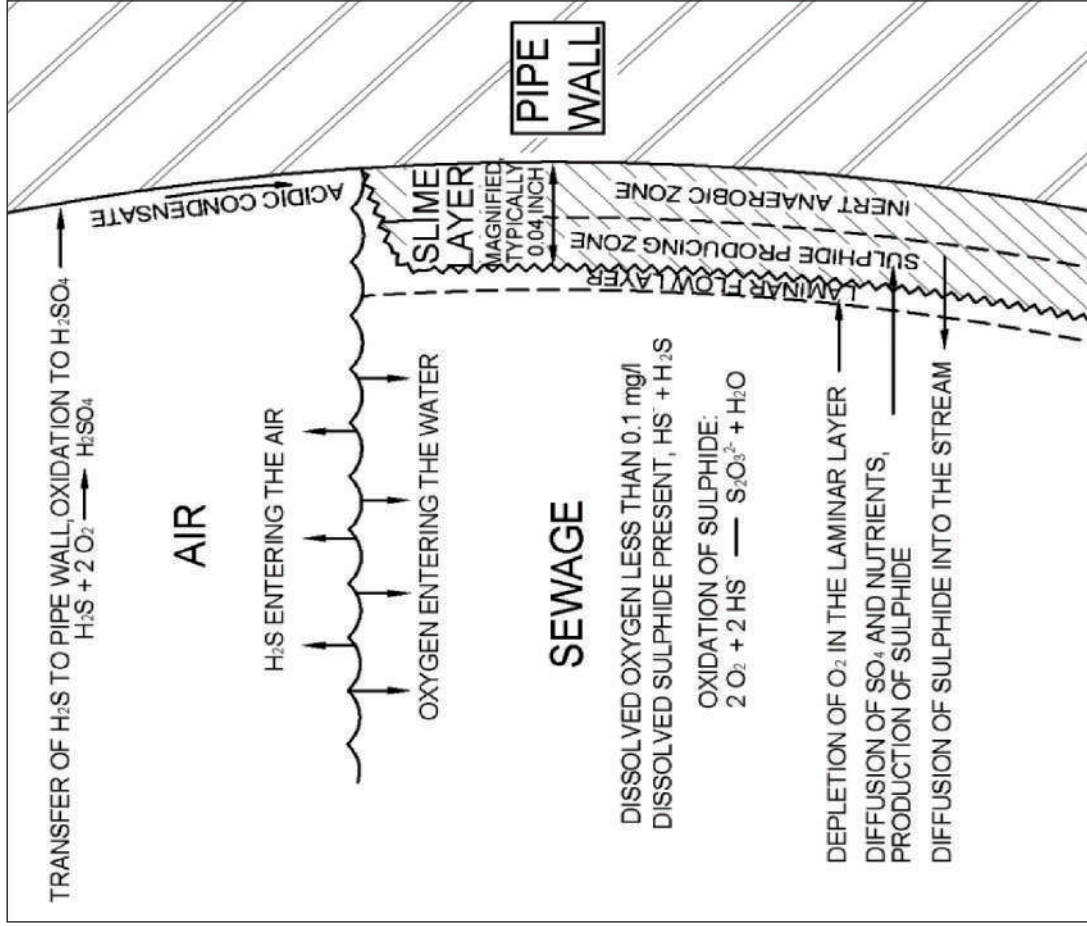
Source: USEPA, 1974

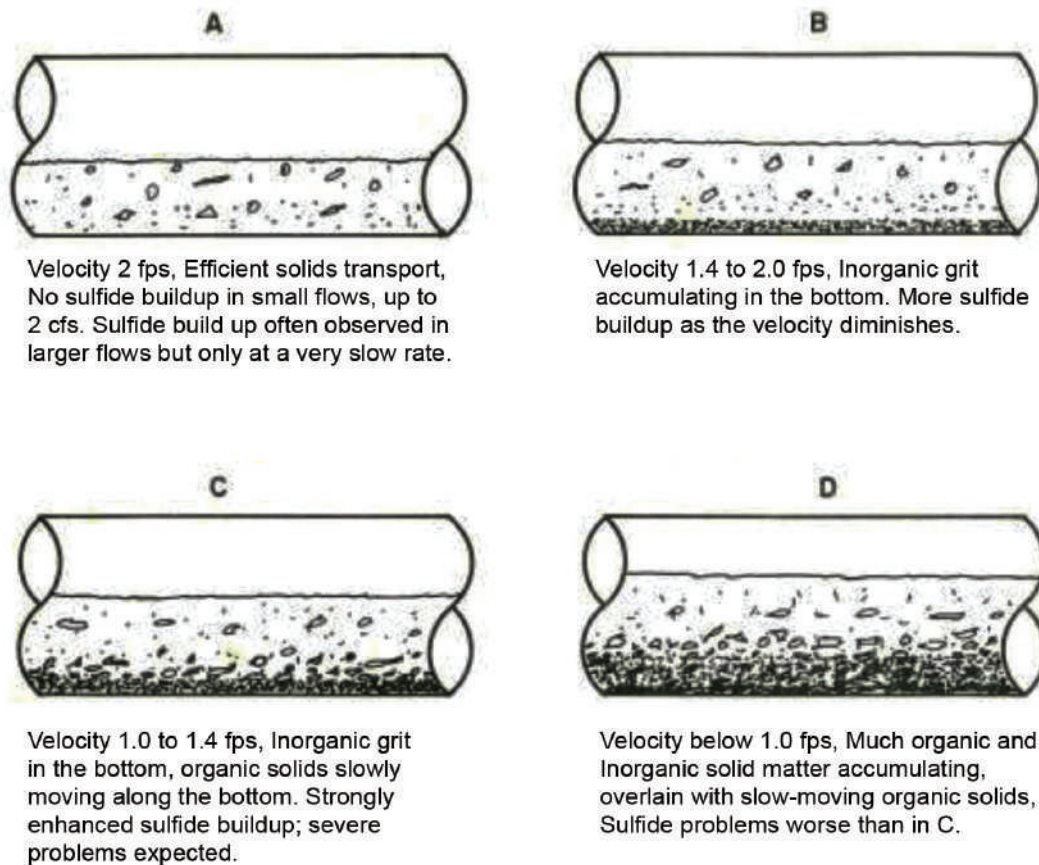Figure 3.56  Sulphide gas equilibrium in negligible oxygen
conditions in sewers
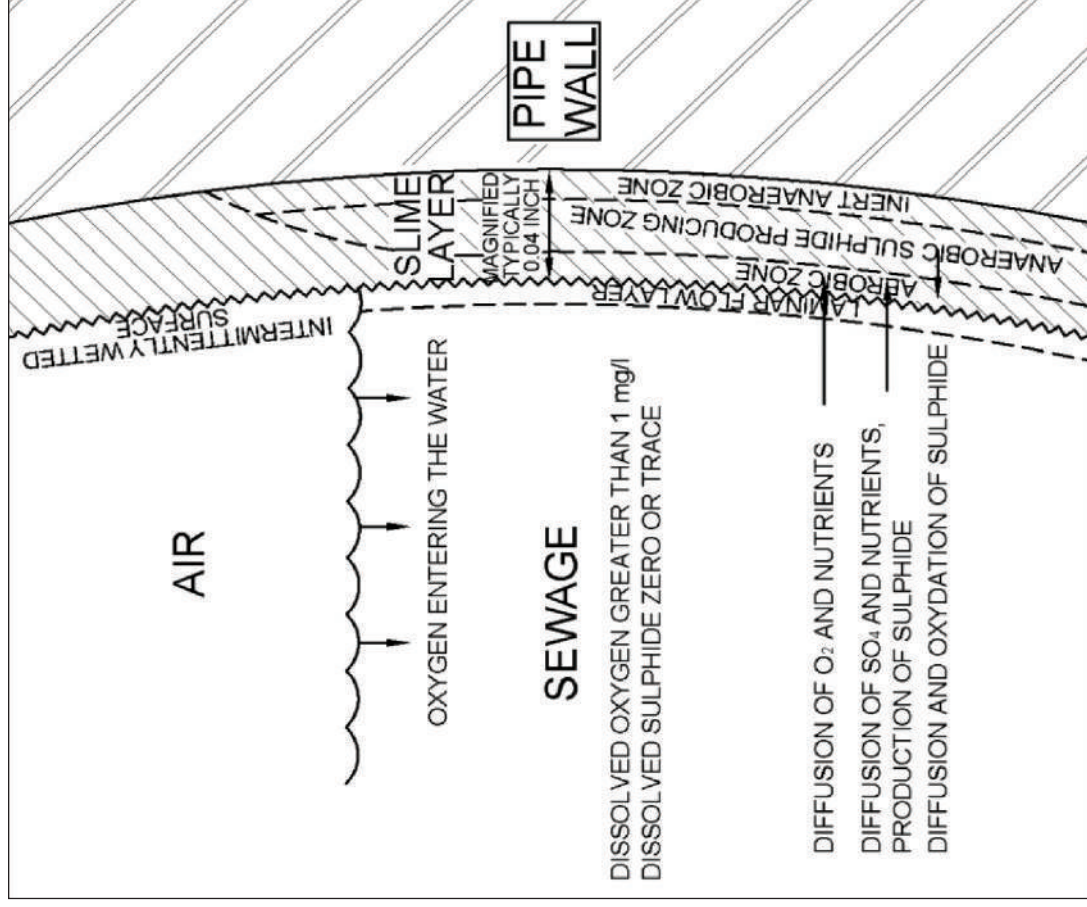


Source: USEPA, 1974

Figure 3.57  Sulphide gas equilibrium in appreciable oxygen
conditions in sewers

**3.69.4.3      Velocity of Flow**

The velocity should be both self-oxidizing and self-cleansing. If the velocity of flow is great enough to keep the submerged surfaces of the sewer free from slimes, no generation of $H_2S$ will occur. Whenever the velocities are too small, the organic materials get settled out and undergo anaerobic decay and release the sulphide, which later combines with the moisture and forms sulphurous and sulphuric acid. The effect of velocity and relative sedimentation of organics and grit is shown in Figure 3.58.



**A**

Velocity 2 fps, Efficient solids transport, No sulfide buildup in small flows, up to 2 cfs. Sulfide build up often observed in larger flows but only at a very slow rate.

**B**

Velocity 1.4 to 2.0 fps, Inorganic grit accumulating in the bottom. More sulfide buildup as the velocity diminishes.

**C**

Velocity 1.0 to 1.4 fps, Inorganic grit in the bottom, organic solids slowly moving along the bottom. Strongly enhanced sulfide buildup; severe problems expected.

**D**

Velocity below 1.0 fps, Much organic and Inorganic solid matter accumulating, overlain with slow-moving organic solids, Sulfide problems worse than in C.

Source:USEPA,1974

Figure 3.58  Solids accumulation at invert of mains at various velocities

This incidentally brings out the fact that under prolonged conditions of absence of the minimum velocities and the absence of the high velocities associated with the peak flow conditions at least once a day, the effective area of the pipe is progressively reduced. This affects the gravity sewers by backing up of the sewage upstream and reduced pumping in pumping mains resulting from higher heads needed to pump the same volumes of sewage in reduced bores of the pipe.

The velocity necessary to prevent the build-up of sulphides in flowing sewage corresponding to different values of the effective BOD ($BOD_T$) are shown in Section 3.15.2.

In determining the velocity to be used in design, the effective BOD should be calculated for the period of the year, which gives the maximum value.

**3.69.4.4    Age of Sewage**

The oxidation-reduction potential of sewage which in turn is influenced by the age of sewage, seems to be one of the important factors contributing to sulphide build up in the lower reaches. When septic sewage is discharged from a collecting system, an Imhoff tank, or from a septic tank into an outfall, it should be treated before it goes into the sewer. When outfall grades are steep, the problem is particularly acute since high turbulence can release the sulphides causing odour and corrosion problems. Long detention times in forced mains greatly influence the generation of sulphides.

The possible sulphide build up in a filled pipe can be roughly estimated as:

$$\Delta C_s = 0.066\, t\, BOD_T\, \frac{(1 + 0.0004\, d)}{d} \tag{3.47}$$

where,

$C_s$ :   Increase of Sulphide concentration in the force main in mg/l

$t$ :   Detention time in the main in minutes

$d$ :   Pipe diameter in mm

**3.69.4.5    Hydrogen Ion Concentration**

Sulphide producing organisms are known to have a considerable adaptability so that pH value is not likely to have much effect on the rate of generation in sewers within the pH of 6 to 8. If the pH value is above 9.0 or below 5.5, sulphide generation will be affected.

**3.69.4.6    Sulphate Concentration**

The more the concentration of Sulphate, the more is its reduction to $H_2S$.

**3.69.4.7    Ventilation**

Ample ventilation through sewers will help in carrying away the generated $H_2S$, supply additional oxygen to the sewage and keep the walls free of moisture and reduce the tendency for sulphuric acid formation and attack of concrete.

Ventilation is particularly important in locations of turbulent flow, either by better natural ventilation or by forced ventilation by fans, one or more of the necessary factors for optimal bacterial activity can be made limiting. However, it is often very difficult and expensive to provide enough ventilation to prevent corrosion.

**3.69.5    Sulphide Control Procedures**

The following are some of the criteria that may be taken into account in preventing or controlling sulphide build up and consequent odour and/or corrosion.

### 3.69.5.1    Design of Sewers for Sulphide Corrosion Issue

In the design of sewer systems, consideration should be given to the desirability of maintaining velocities sufficient to avoid sulphide build up and of minimizing pressure lines and points of high turbulence. The designer should take into consideration topography, grades of sewers, ventilation, materials of construction, sewage temperature and strength, etc.

Some of the design features that should be considered are described below.

One of the important factors in the control of $H_2S$ is the velocity of flow and BOD. Please refer Sections 3.15.2 for BOD to prevent $H_2S$ in sewers. The limiting velocities for prevention of sulphide generation vary with temperature and effective BOD. The velocities given in Section 3.15.1 are believed to be the minimum that should be used. An allowance of 25% in the velocity should be made as a factor of safety and if industrial wastes are present with a higher content of dissolved organic matter, it may be necessary to increase this allowance to 50%. Where it is impractical to provide a sewer gradient in design to give these limiting velocities, other means of controlling sulphide generation should be considered. Velocities giving high, single point turbulence, may however, result in sulphide release and severe odour and/or corrosion.

Except in the cases where sewage is quite weak and in a fairly well aerated condition, high sulphide generation because of large slime areas can be expected in completely filled sewer lines. Force mains, therefore should be kept to a minimum or velocities must be adequate at all flowing times.

Since biological activity is concentrated largely in the slime layer, it increases with an increase of the wetted perimeter. The oxygen uptake is proportional to the surface width of the stream. Therefore, it follows that deep flow in a pipe is more conducive to sulphide generation than shallow flow. Accordingly, where sulphide generation is a critical consideration, a larger pipe is always better than a smaller one for any given slope and sewage flow.

Turbulence caused by high velocities for short distances or improper design of junction manholes permitting sewage lines to intersect at right angles or at different elevations should be avoided as turbulence can cause excessive release of $H_2S$ even where sewage contains only a small amount of dissolved sulphides.

Concrete with a low water-cement ratio of suitable workability, thorough mixing, proper placing and sufficient curing is preferred for sewers.

### 3.69.5.2    Control of Sewage Character for Sulphide Corrosion Issues

Trade wastes containing dissolved sulphides should not be allowed into the sewers. High sulphate concentrations arising from the discharge of tidal or sea-water to the sewer should be controlled. The oxidation-reduction potential of the sewage can be increased and the rate of generation of $H_2S$ slowed down by steps, which include the partial purification of sewage allowed into the sewers by sedimentation or by high rate treatment on filters. Effective BOD of sewage depends upon sewage strength and temperature. By reducing sewage strength and/or temperature, effective BOD as well as minimum velocity required can be reduced.

Strength of sewage can be reduced in some cases by diluting sewage with unpolluted water. It must be realized, however, that dilution reduces the waste-carrying capacity of the sewer.

Where velocities are inadequate to control the formation of $H_2S$ or where completely filled lines are encountered as in force mains, supplemental aeration by the use of compressed air may be desirable. Air injection would prevent hydrogen sulphide building up and in any case will greatly reduce generation.

Air addition at about 10 lpm for each cm of pipe diameter is necessary. Care must be taken to prevent the formation of air pockets in such lines, since experience has shown that some $H_2S$ will form on the walls at the points of such air pockets and corrosion will occur.

### 3.69.5.3    Cleaning of Sewers for Sulphide Corrosion Issues

Removal of slime and silt has the effect of reducing sulphide generation. Periodic cleaning of sewers by mechanical or chemical means is necessary. Any partial blocking of the sewer by debris will result in retardation of flow and consequent anaerobic decomposition of deposited sludge. Periodic mechanical cleaning and flushing of sewers can reduce average sulphide generation by 50 %. A good continuing programme of mechanical cleaning is probably the foundation for any control programme.

Sulphuric acid is effective in reducing slimes. Intermittent use of sulphuric acid was found to be useful in removing slimes on the submerged walls. Caution must be exercised in the use of sulphuric acid for the purpose of acidification since iron sulphide, that may be present on sewer walls, may cause an initial release of $H_2S$ sufficient to be fatal to any worker inside the sewer. The shift of pH value also changes all the ionized sulphide (in the flow) to $H_2S$.

Slaked lime, $Ca(OH)_2$ is probably more suitable for chemically treating the slime since no corrosion damage will result from it and sulphide release will not occur. It has been found that if the slimes are subject to lime slurry of about 8,000 mg/L for 45 minutes, they will be inactivated for periods of 3 to 14 days depending upon flow and sewage characteristics.

### 3.69.5.4    Chlorination for Sulphide Control Issues

Chlorine has been successfully used in controlling sulphide generation for many years. Chlorine is effective in three ways (i) it destroys sulphides by chemical reaction, (ii) it reduces biological activity and produces mild oxidizing compounds in the sewage and (iii) it destroys the slimes.

An approximate dosage of 10 to 12 mg/L of chlorine is sufficient. When excess chlorine is applied, it leaves the sewage in an oxidized state, and prevents the re-appearance of sulphide for some distance downstream.

### 3.69.6    Materials of Construction for Sulphide Corrosion Issues

When corrosion cannot be prevented by design, maintenance or control of wastes entering the sewer, consideration must be given to corrosion resistant materials such as vitrified-clay or to protective linings of proven performance.

Plastic pipes may also be used if accepted in all other respects. It is possible that super sulphated metallurgical cement, pozzolana-portland cement mixtures or portland cement low in tricalcium aluminate may be more resistant to attack than normal portland cement.

On concrete pipe, extra wall thickness (sacrificial concrete) sometimes is specified to increase pipe life in the event corrosive conditions develop. On reinforced concrete, this takes the form of added cover over the inner reinforcing steel.

Another method of modifying the composition of concrete is by the use of limestone or dolomite aggregate in the manufacture of the pipe materials. The use of such aggregates increases the amount of acid-soluble material in the concrete, which prolongs the life of the pipe in corrosive environments. The rate of acid attack of limestone or dolomite aggregate pipe may be only about one fifth as great as when granite aggregate is used.

Unfortunately, not all limestone and dolomite aggregates exhibit the same resistance to this form of corrosion. Accordingly, tests should be made before limestone or dolomitic aggregate is used. Aluminous cement has initial resistance to acid attack. Its corrosion products are also not extensive. Therefore, it may have limited use in sewer structures.

### 3.69.7    Sewer Protection

Protection of sewer structures by lining or coating against $H_2S$ attack can also be considered if other methods of control are impracticable.

### 3.69.7.1    Liners

A plastic polyvinyl chloride sheet, having T-shaped protections on the back, which key into the pipe wall at the time of manufacture, is one of the successful lining materials. Vitrified clay of low porosity has also been used as a liner. In regions where high sulphides and high production of $H_2SO_4$ can be expected, the problems remain.

Cement mortar joints are subject to attack. Bituminous joints are emulsified and dissolved by soaps, oil and grease. Acid-proof cement joints offer the best protection, but they are costly. Some type of plastic coatings and/or linings for sewers and other structures have proved moderately successful, given continued inspection and maintenance.

The function of these linings is to isolate the concrete from the corrosive atmosphere. To be effective, the lining including joints must be sealed completely to protect the sewer system throughout its expected life.

The interior of cast iron and ductile iron pipe usually is lined with cement mortar. Steel pipe sometimes is lined similarly. Smooth-walled steel pipe also may be protected by cementing plasticized polyvinyl chloride sheets to the pipe and sealing the joints.

Corrugated metal pipe may be coated inside and out with bituminous material. For added protection, asbestos fibres may be embedded in the molten zinc before it is bituminous-coated (asbestos bonded). Such coatings should be of impermeable material of sufficient thickness and free of flaws such as pin-holes.

### 3.69.7.2    Protective Coatings

Any protective coating used should possess the following qualities; (i) it should be resistant to acid attack, (ii) it should bond securely to the concrete, (iii) it should be economical and durable, (iv) it should be resistant to abrasive action by flow of sewage, and (v) when applied, it should be thin enough to fill all pores and irregularities in the surface. The coating should be continuous with no pin holes or other breaks. Figure 3.59 presents a RCC sewer pipe with coating.



Figure 3.59  RCC Sewer pipe with protective coating

The effectiveness of a coating thus depends on its inherent resistance to acid attack and on its ability to form impervious membrane. In practice, no coating can be applied without discontinuity. Inspection and maintenance must be periodical.

Plastic-based paints and coal tar epoxy coatings have proved to be good.
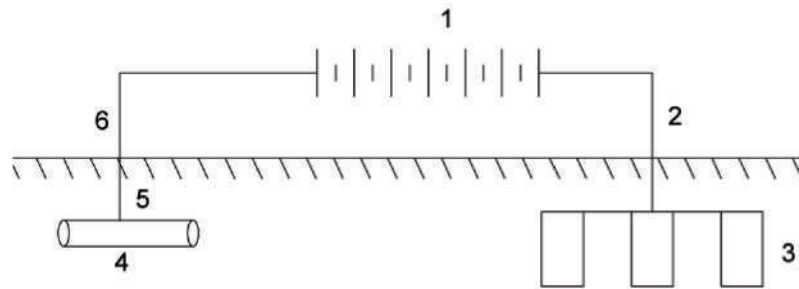
### 3.69.7.3    Cathodic Protections

Cathodic protection is the application of electricity from an external power supply or the use of galvanic methods for combating electrochemical corrosion.

Cathodic protection should be used as a supplement and not as an alternative technique to other methods of protection. It may be a more suitable and expeditious method of protection for existing pipelines.

### a)  Basic Principle

The basic principle is to make the entire surface of the equipment cathodic, thus affording protection, since corrosion takes place only at the anodic surface. This can be achieved by connecting it to a DC source. In this case, the anode consists of specially earthed electrodes. The general arrangement in a cathodic protection assembly is shown in the Figure 3.60 overleaf.

Source: CPHEEO, 1993

Figure 3.60  General arrangement of cathodic protection

The current from the positive pole of the DC source flows through the conductor 2 into the earthed anode 3 and then into the soil. From the soil the current flows to the surface of the pipe 4 to be protected and flows along the pipe to the drainage junction point 5, the conductor 6 and back to the negative terminal of the current source. Thus, the entire surface of the underground pipe or equipment becomes cathodic and is protected from corrosion, while the earthed anode gets corroded. The anode, is usually scrap metal e.g., old tubes, rails, etc. Other metals, which are resistant to attack by surrounding soil like special alloys or graphite, are also used. The conductivity of the protective coating has a direct influence on the length of the protected section of the pipe. The required power increases with increasing conductivity of the coating.

## b)  Preliminary Investigations

The existing pipeline has to be inspected to ascertain the sections that require protection. Other basic information required are as follows:

a) Plan and details of the pipelines (showing branch connections, diameter, length and wall thickness) and location plan of the section to be protected along with

 i) Data on soil resistance along the section to be protected at the intervals of at least 100 m as well as the earthing points

ii) Information on the availability of sources of electricity, amperage, voltage, DC/AC (phase) in the vicinity and spaces for housing current supply and controls

iii) Data on the conductivity or resistivity of the existing protective insulation; and

iv) Condition of the pipeline, if it is already in use

## c)  Power requirements

With the above data, minimum current density and maximum protection potential can be worked out. The capacity of the current source for a cathodic protection system depends on (1) length of the section to be protected (2) type and state of the coating of the pipeline (3) diameter of the pipe (4) wall thickness of the pipe (5) conductivity of the soil and (6) design of anode earthing. The power requirements vary from 0.4 to 10 kilowatts in most cases. The possible current sources are DC Generator, converter-rectifier, storage batteries of dry or acid type. The pipeline should be at least 0.3 V negative to the soil.

## d) Anodes

The main power loss occurs in the anode earthing. The earthing can be carried out by any metal (pure or scrap) of any shape and carbon forms like coke or graphite. When tubes are used, the earthing can be either horizontal or vertical. Near the earthing zone, soil treatment can be done to reduce soil resistance by adding salts like sodium chloride, calcium chloride or moistening the soil, the former being better and long lasting. Carbon or graphite electrodes have longer durability than metal electrodes.

## e) Other facilities

A cathodic protection station should provide space for housing the equipment, installation of current sources, supply and distribution zones, equipment for check measurements, construction of earthing structures and facilities for carrying out operational tests.

### 3.69.7.4    Protection by Sacrificial Anode

Sacrificial anodes serve the same purpose as the cathodic protection system but do not require external electric power supply. The required current is supplied by an artificial galvanic couple in which the parts to be protected, usually iron or steel, is made as the cathode by choosing the other metal having the higher galvanic potential, as the anode. Zinc, Aluminium and Magnesium (with sufficient purity) or their alloys, which are higher up in the galvanic series must be used for this purpose. Sheets of zinc suspended in a coagulation basin are an example. A single protector anode will not be sufficient and it will be necessary to install a number of such anodes generally spaced at 4 to 6 m in the pipeline or the structures to be protected.

The performance and service life of anodes depend mostly on the nature of soil or water surrounding them. Use of fill materials in the soil such as clay and gypsum powder results in low resistance of anode earthing and yields a high current. The costs of protection by galvanic anode would be appreciably higher in the case of pipeline networks in big towns since it would be necessary to suppress incidental contacts.

For the application of galvanic protection, the resistance of the soil should be less than 12,000 ohm-cm. A higher resistance of the circuit can achieve neither the required current density, nor the reduction of the pipe to soil potential. In such cases, cathodic protection by means of external power supply offers better protection.

The following measures are also of interest in minimizing corrosion:

i)  Minimizing point of high turbulence within the system thus resulting in less sulphide generation

ii) Designing wet wells to preclude surcharge of tributary lines which also result in less sulphide generation

iii) Provision of forced ventilation at a point where air may be depleted seriously of its oxygen

iv) Using a coating of another metal such as zinc, galvanized iron or using paints appropriately

v) Gas Scrubbing

vi) Providing inside sleeving or lining of suitable type of plastic materials

The problem of sewer corrosion due to hydrogen sulphide production and its control is a serious one to the sewage conveyance system. Prevention of $H_2S$ generation by proper design and continued cleaning of sewers seems to be the best available method.

## 3.70   CONNECTION OF HOUSE SEWER TO PUBLIC SEWER

The earlier practice has been to connect house sewers to public sewers using the typical Y branch or T branch depending on the depth of public sewer. The reason for this is that stoneware pipes had these specials and can be inserted wherever needed while laying the sewers. Most of the problems of sewer blocks are traceable to solid materials getting stuck at "T" or "Y" junction in house services, requiring most times even cutting open the roads. It is henceforth proposed to discontinue this practice. The house-service sewer connections shall be effected only in manholes. In case of old sewers, a new manhole shall be inserted for this purpose. The material of the house service sewer shall be either conventional salt glazed stoneware or UPVC rigid straight pipes of 6 kg/cm2 pressure class in manufacture and as per IS 15328 with solvent cement joints.

The minimum earth cover above the crown of the sewer shall be mandatorily 90 cm and where this becomes impossible; the property owner shall be directed to depress his terminal chamber to comply with this minimum earth cover of 90 cm, as the public manhole shall start at its crown at 100 cm below ground level (see also section 3.20.3). Where such sewers cross the electricity power cables, the specifications of IS 1255 clause 6.3.3 and clause 6.3.3.1 shall be mandatorily followed without any exception. All such house service sewers shall be only above the electricity power cable and the minimum clearance shall be 30 cm all-round the electricity power cable. The electric power cable itself shall be covered all around by 15 cm riddled soil and further protected on top by tiles, bricks or slabs. Hence, the total minimum clearance will be 30 cm + 15 cm = 45 cm.

The house owner shall be mandated to possess a "kraite" a type of non-corroding, sufficiently flexible but rigid type of less than 10 mm diameter rod, which he/she shall use to rod the house-service sewer freely up to the manhole. The labour of the local body shall not be deployed for any removal of obstructions in the house-service sewer. Typically, it is possible to effect six service connections to a manhole. Without exception, the provision of terminal chamber inside the property premises shall be mandatorily followed.

## 3.71   SPACING OF MANHOLES

Sewers are known to get choked resulting in sewage overflow from upstream manholes. The non-invasive and non-destructive cleaning is by equipments like jetting machines, bucket machines, rodding machines etc. At the same time, however, house-service sewers are also known to get choked and sewage backs up into the houses. The reasons in this case are

(a) the choking of public sewers and manholes in road portion or

(b) obstructions in the house sewer itself due to extraneous material pushed into it by the residents.

The problem at

(a) can be relieved when the public sewer is cleared up.

(b) however requires clearing the house-service sewer.

In the older practice of house service sewers joining the public sewer through "Y" or "T" junctions, this is difficult and invariably, the road is dug up at the junction to break the house service sewer, clear it up and join back by covering with a curved tile or sleeve etc. Even then, the choking can recur and the practice is to be repeated resulting in the weakening of the service sewer itself to withstand the load from the road.

The Chennai Metropolitan Water Supply and Sewerage Board (CMWSSB) have been successfully implementing for over two decades, the practice of connecting all house service sewers to manholes and not public sewers. The precaution used is such connections are effected below the corbel portion of the manholes. Typically, a manhole takes six such connections, three from each side of the road, from properties opposite to and on both sides of the manhole. The clearing of the house-service itself seldom arises because even the extraneous matter pushed in by the residents gets "dropped" into the manhole and if at all noticed, the simple rodding of the house sewers by bamboo splits or flexible rods succeeds in clearing the blockages and drop them into the manholes. Thus, the problem of the road getting dug up frequently causing nuisance in the public is completely avoided.

From this point of view, it becomes necessary to limit the length of such house service sewers. Considering that typically properties are developed on plots of about 10 m width, a spacing of 30 m between manholes permits both the objectives of easy cleaning of public sewer stretches and eliminating the avoidable road digging to clear obstructions in house service sewers. Accordingly, the spacing of manholes shall be retained at 30 m as in the existing manual and in case of economically weaker sections, the spacing can be narrower, commensurate with the width of the plots so that all connections are made to the manholes without undue lengths.

In addition, there will be additional manholes at changes of directions and in the case of commercial structures like meeting halls, marriage halls etc as the case may be. Very wide plots should be encouraged to avail manholes at each end of the plot by meeting the cost of the extra manhole. In the case of gravity outfall sewers with no house service sewers, the spacing can be at 60 m besides at every change of direction and drops. For insertion of the house service sewers into the manholes, it is necessary to have a precast ring section below the corbel portion with holes at 45 degrees to the public sewer line to facilitate insertion of three house service sewers on each side of the road.

Usually the house service sewers shall be 110 mm or 160 mm UPVC 4 kg / sqcm (as detailed in sewer laying section). Accordingly, the height of the ring shall be 250 mm and 300 mm to permit filling of the annular the interspaces between the sewer and the opening with cement concrete and at least 50 mm of RCC annular fill around the inserted house service sewer respectively. Without exception, the provision of terminal chamber inside the property premises shall be mandatorily followed.

### 3.71.1    In Public Sewers

The CMWSSB are connecting all the house service sewers directly below the corbels of manholes for over two decades. Typically, a manhole takes six such connections, three from the properties on each sides of the public sewer alignment. The clearing of the house service itself seldom arises because even the extraneous matter pushed in by the residents gets "dropped" into the manhole. Where needed, simple rodding of the house sewers by bamboo splits or flexible rods from the terminal chamber of the property clears the blockages and drop it into the manholes. Thus, the problem of the road getting dug up is completely avoided. Such a system is also referred to as the "Berlin System" as in Figure 3.61 as cited by the EPA in its publication quoted here.
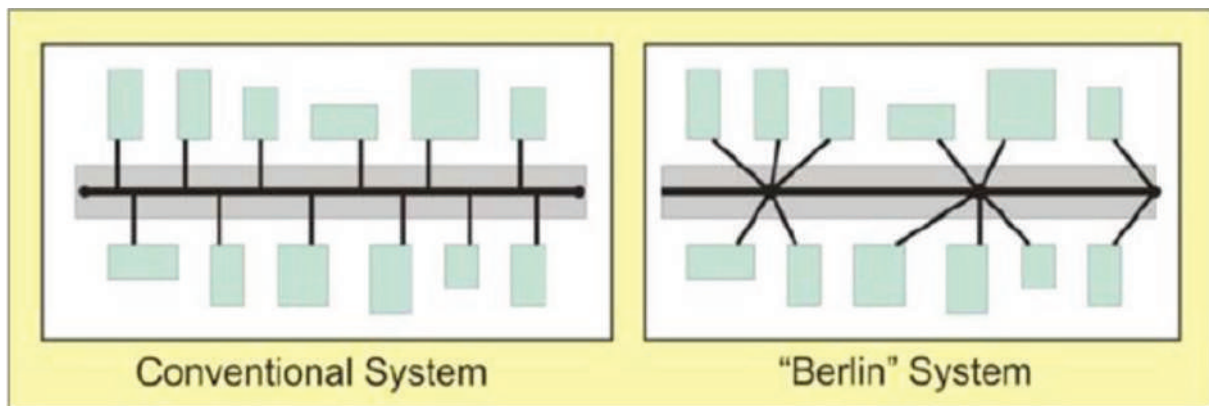


Source: EPA, 2010

Figure 3.61  Connection system between house service sewer and public sewer

Considering that the typical road frontage of plots in ULBs at about 10 m and the road widths being two lanes with 6 m to 8 m width, a spacing of 30 m between manholes permits restricting the lengths of house sewers between 3 m in the shorter perpendicular direction and 15 m in the longer hypotenuse direction. These are reasonable for maintenance and cleaning. Accordingly, the spacing of manholes shall be 30 m. In the case of economically weaker sections, the spacing can be lesser,  commensurate with the width of the plots. The standard provision of additional manholes at changes of directions will continue. In the case of meeting halls, marriage halls etc as also very wide plots these should be encouraged to avail manholes at each end of the plot by their meeting the cost of the extra manhole.

### 3.71.2   In Outfall or Trunk Gravity Sewers

In outfall or trunk gravity sewers with no house service connections, the diameter of the sewers will also be larger as 1.5 m in diameter. The Japanese manual specifies as in Table 3.30 overleaf.

The WEF states the spacing of manholes as "Manholes are provided in sewer systems to help maintain and clean sewer pipes. Typically, they are provided at intersections of two or more mainline sewers, at changes in direction of sewer lines, and at regular intervals along a mainline. Manholes are typically spaced approximately 300 feet (91.4 meter) apart, but can be less than 100 feet (30 meter) or as far apart as 500 feet (152 meter) (EPA)".

Table 3.30  Spacing of manholes as per Japanese Manual

| Sewer Diameter (mm) | 600 or less | 1000 or less | 1500 or less | 1650 or more |
|---|---|---|---|---|
| Maximum Manhole Space (m) | 75 | 100 | 150 | 200 |

Source: JSWA

Considering the involved aspects of costs, frequency of cleaning and financial sustainability of sewer cleaning equipment by the ULBs, the following in Table 3.31 is now recommended. It is always possible to insert a manhole later on in between the existing manholes if another sewer or house service sewer is to be connected at that time.

Table 3.31  Spacing of manholes in gravity sewers not receiving house service sewers

| Sewer Diameter (mm) | Up to 600 | 600 to 900 | 900 to 1200 | 1200 to 1500 |
|---|---|---|---|---|
| Maximum Manhole Space (m) | 60 | 90 | 120 | 150 |
| Maximum Manhole Space (m) | 30 m or it can be less than 30 m depending on the street | | | |

Where sewer diameters exceed 1500 mm, the possibility of using egg shaped sewers made out of pre-cast RCC sections made out of sulphate resisting cement and duly plastered on both sides should be explored for easier execution of work and better control over the flushing and cleaning at low flows in the bottom egg shaped segment. The geometry of the egg shaped sewer and its hydraulic properties at full flow are shown in Figure 3.62. The characteristics of flow shall be referred to from standard texts.
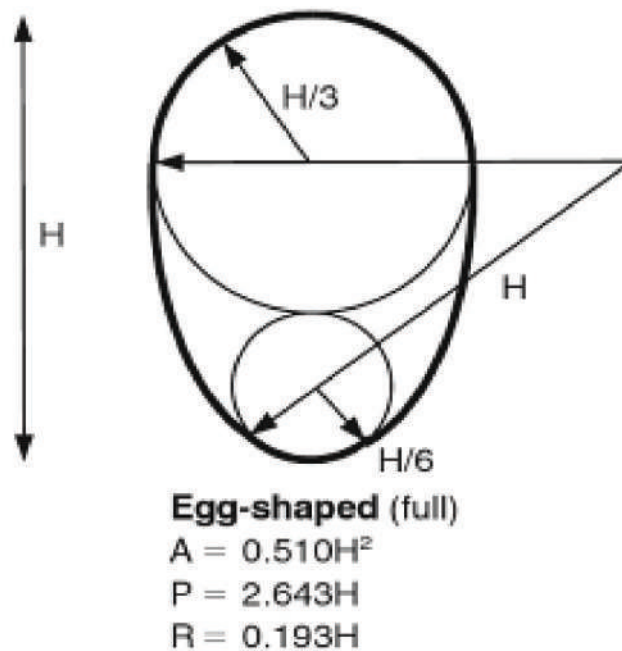


**Egg-shaped** (full)
$A = 0.510H^2$
$P = 2.643H$
$R = 0.193H$

Figure 3.62  Geometry and hydraulics of egg shaped sewer at full flow

## 3.72    LIFT STATIONS IN GRAVITY SEWERS

There are cases where high water table conditions or rocky strata pose considerable difficulties in the design and provision of conventional gravity sewerage system in that excavations amidst sub soil water or rocky terrain is not only difficult but also is frowned upon by the public when the works drag on and on in the middle of the road. Such situations can be easily got over by restricting the depth of sewers to a practicable limit and diverting the flow into a pavement submersible pump station with a lockable control panel there itself. This is similar to the pillar boxes of the electricity board and the delivery main can lift the flow to the downstream manhole at the conventional 0.9 m depth to invert.

With the availability of quite a few manufacturers of sewage-submersible pump sets in the country it should be possible to implement this instead of struggling with deep sewers in such areas for years together and more importantly compounding the problems of O&M and the repairs at these depths perpetually. These submersible pump stations can be operated by mercury float switches and powered by dedicated feeder lines from the local electrical authority similar to the lines given to the hospitals, etc. These pump sets can also be connected to solar panels. The pump pit can be covered with pedestrian grade walkway slabs, which are of RCC and with adequate lifting arrangements to permit the lowering and lifting the submersible pump sets. More details on lift stations are available in Chapter 4.