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.

3.3.1 Rational Method

3.3.1.1 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 the expression,

$$Q = 10\ C\ i\ A \qquad (3.1)$$

Where Q is the runoff in m^3/hr;

`C' is the coefficient of runoff;

`i' is the intensity of rainfall in mm/hr and

`A' is the area of drainage district in hectares.

3.3.1.2 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:

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

3.3.1.3 Intensity of Precipitation

The intensity of rainfall decreases with duration. Analysis of the observed data on intensity 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 12mm/hr to 20mm/hr.

Table 3.1 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 3.1
ANALYSIS OF FREQUENCY OF STORMS

Duration in Minutes	Intensity mm/hr	30	35	40	45	50	60	75	100	125
		No.of Storms of 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							

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 and given in Table 3.2.

TABLE 3.2
TIME INTENSITY VALUES OF STORMS

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

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

i) $$i = \frac{a}{(t^n)} \qquad (3.2)$$

ii) $$i = \frac{a}{t + b} \qquad (3.3)$$

Where,

i	=	intensity of rainfall (mm/hr)
t	=	duration of storm (minutes) and

a,b and n are 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, (t_c).

3.3.1.4 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. Time of concentration (t_c) is equal to inlet time (t) plus the time of flow in the sewer (t_f). 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.

3.3.1.5 Coefficient of Runoff

The portion of rainfall which finds its way to the sewer, is dependent on the imperviousness and the shape of tributary area apart from the duration of storm.

a) ***Imperviousness***

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

Type of area	Percentage of Imperviousness
Commercial and Industrial area	70 to 90
Residential Area:	
i) High density	60 to 75
ii) Low density	35 to 60
Parks & undeveloped areas	10 to 20

The weighted average imperviousness of drainage basin for the flow concentrating at a point may be estimated as

$$I = \frac{A_1 I_1 + A_2 I_2 \ldots}{A_1 + A_2 + \ldots} \qquad (3.4)$$

Where,

A_1, A_2 = drainage areas tributary to the section under consideration

I_1, I_2 = imperviousness of the respective areas and

I = weighted average imperviousness of the total drainage basin.

b) ***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.

c) ***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. Runoff from an area is significantly influenced by the saturation of the surface 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.

d) ***Computation of Runoff Coefficient***

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 times of concentration are given in Table 3.3. Although these are applicable to particular shapes of 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.

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

TABLE 3.3
RUN OFF COEFFICIENTS

Duration, t, minutes	10	20	30	45	60	75	90	100	120	135	150	180
Weighted Average Coefficients												
1) Sector concentrating in stated time												
(a) Impervious	.525	.588	.642	.700	.740	.771	.795	.813	.828	.840	.850	.865
(b) 60% Impervious	.365	.427	.477	.531	.569	.598	.622	.641	.656	.670	.682	.701
(c) 40% Impervious	.285	.346	.395	.446	.482	.512	.535	.554	.571	.585	.597	.618
(d) Pervious	.125	.185	.230	.277	.312	.330	.362	.382	.399	.414	.429	.454
2) Rectangle (length = 4 x width) concentrating in stated time												
(a) Impervious	.550	.648	.711	.768	.808	.837	.856	.869	.879	.887	.892	.903
(b) 50% Impervious	.350	.442	.499	.551	.590	.618	.639	.657	.671	.683	.694	.713
(c) 30% Impervious	.269	.360	.414	.464	.502	.530	.552	.572	.588	.601	.614	.636
(d) Pervious	.149	.236	.287	.334	.371	.398	.422	.445	.463	.479	.495	.522

3.4 HYDRAULICS OF SEWERS

3.4.1 Type of Flow

Flow in sewers is said to be steady, if the rate of discharge at a point in a conduit remains constant with time, and if the discharge varies with time, it is unsteady. If the velocity and depth of flow are the same from point to point along the conduit, the steady open channel flow is said to be uniform flow, and non uniform if either the velocity, depth or both are changing. In laminar flow the fluid moves along in smooth layers, while in turbulent flow the fluid moves in irregular paths.

The hydraulic analysis of sewers is simplified by assuming steady flow conditions. In large storm channels, or where surge or water hammer is predominent, as in pumping mains, the flow can be unsteady. Most sewers have turbulent flows with stream lines following the boundaries.

A properly functioning sewer has to carry the peak flow for which it is designed and transport suspended solids in such a manner that deposits in a sewer are kept to a minimum. The design for waste water collection system presumes flow to be steady and uniform. The unsteady and non-uniform waste water flow characteristics are accounted in the design by proper sizing of manholes.

3.4.2 Flow - Friction Formulae

The available head in waste water lines is utilised in overcoming surface resistance and, in small part, in attaining kinetic energy for flow.

Estimated design flows depend to a large extent on the assumptions, the accuracy of which is variable. Inspite of this, care is required to select an accurate friction flow formula as to avoid compounding errors. However, the design practice is to use the Mannings formula for open channel flow and the Hazen Williams and Darcy-Weisbach formulae for closed conduit or pressure flow.

3.4.2.1 Mannings Formula

$$V = [(1/n)] \times [R^{2/3} \; S^{1/2}] \qquad (3.5)$$

For circular conduits

$$V = (1/n) \; (3.968 \times 10^{-3}) \; D^{2/3} \; S^{1/2} \qquad (3.6)$$

$$\text{and } Q = (1/n) \; (3.118 \times 10^{-6}) \; D^{2/3} \; S^{1/2} \qquad (3.7)$$

Where

Q	=	discharge in lps
S	=	slope of hydraulic gradient
D	=	internal dia of pipe line in mm
R	=	hydraulic radius in m
V	=	velocity in mps
n	=	Manning's coefficient of roughness

A chart for Manning's formula is given in Appendix 3.2.

The values of Manning's coefficient for different pipe materials are given in Table 3.4.

A reduction in the value of `n' has been reported with increase in diameter.

3.4.2.2 Darcy Weisbach Formula

Darcy and Weisbach suggested the first dimension - less equation for pipe flow problems as,

$$S = \frac{H}{L} = \frac{fV^2}{2gD} \qquad (3.8)$$

Where

H	=	head loss due to friction over length L in meters
f	=	dimension-less friction factor
V	=	Velocity in m/s
g	=	acceleration due to gravity in m/sec^2
D	=	Internal diameter in meters

This formula is not normally used in the design of sewers. Reference may be made to IS 2951 for calculation of head loss due to friction according to Darcy Weisbach formula.

TABLE 3.4

COEFFICIENT OF ROUGHNESS FOR USE IN MANNING'S FORMULA

Type of Material	Condition	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 Socket Spigot Joints (Design Value)		0.011
Masonary	(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) Masonary 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	(a) Unlined	0.013
	(b) With spun cement mortar lining	0.011
Asbestos Cement		0.011
Plastic (smooth)		0.011

3.4.2.3 Hazen-Williams Formula

is expressed as follows:

$$V = 0.849 \; C \; R^{0.63} \; S^{0.54} \quad (3.9)$$

for circular conduits, the expression becomes

$$V = 4.567 \times 10^{-3} \; C \; D^{0.63} \; S^{0.54} \quad (3.10)$$

and

$$Q = 1.292 \times 10^{-5} \; C \; D^{2.63} \; S^{0.54} \quad (3.11)$$

Where,

Q	=	discharge in cum per hour
D	=	internal diameter of pipe in mm
V	=	Velocity in mps
R	=	hydraulic radius in m
S	=	slope of hydraulic gradient and
C	=	Hazen - Williams coefficient

A chart for the Hazen-Williams formula is given in Appendix 3.3.

The values of Hazen-Williams coefficient C for new conduit materials and the values to be adopted for design purposes are furnished in Table 3.5.

3.4.2.4 Friction Coefficients

Friction coefficients for various materials and conditions have been determined based on laboratory and field experiments. Factors which affect the choice of a friction coefficient are conduit material, Reynolds number, size and shape of conduit and depth of flow. Errors inherent in the use of Manning's formula and Hazen-Williams formula are

i) Both formulae are dimensionally inconsistant

ii) The friction coefficients used in the formulae namely Hazen-Williams C and Manning's `n' are usually considered independent of pipe diameter, velocity of flow and viscosity, whereas to be representative of friction conditions these coefficients must depend on relative roughness of pipe and Reynolds Number.

3.4.2.5 Modified Hazen-Williams Formula

The Modified Hazen-Williams formula has been derived from Darcy Weisbach and Colebrook-White equations which overcomes the limitations of Hazen-Williams formula.

The modified Hazen-Williams formula is derived as

$$V = 143.534 \quad C_R \quad R^{0.6575} \quad (S)^{0.5525} \qquad (3.12)$$

in which

V	=	Velocity of flow in mps
C_R	=	Pipe roughness coefficient (1 for smooth pipes, <1 for rough pipes)
R	=	hydraulic radius in m
S	=	friction slope

For more detailed information reference may be made to Chapter 6 of Manual on Water Supply and Treatment.

TABLE 3.5
HAZEN - WILLIAMS COEFFICIENTS

Sl. No.	Conduit Material	Recommended values for	
		New Pipes	Design
1.	Concrete (RCC & PSC) with socket & spigot joints	150	120*
2.	Asbestos cement	150	120*
3.	Plastic pipes	150	120*
4.	Cast iron	130	100
5.	Steel, welded joints	140	100
6.	Steel, welded joints lined with cement or bituminous enamel	150	120*

These pipe materials are less likely to loose their carrying capacity with age, and hence Higher values may be adopted for design purpose if reliable field data is available to justify such revision.

3.4.2.6 Depth of Flow

From considerations of ventilation in waste water flow, sewers should not be designed to run full. All sewers are to be designed to flow 0.8 full at ultimate peak flow. Table 3.6 shows the hydraulic properties of circular sections for Manning's Fromula.

Reference may be made to Fig.3.1 for hydraulic elements of circular sewers and to Fig.3.2 for hydraulic elements of circular sewers that possess equal self cleansing velocity at all depths.

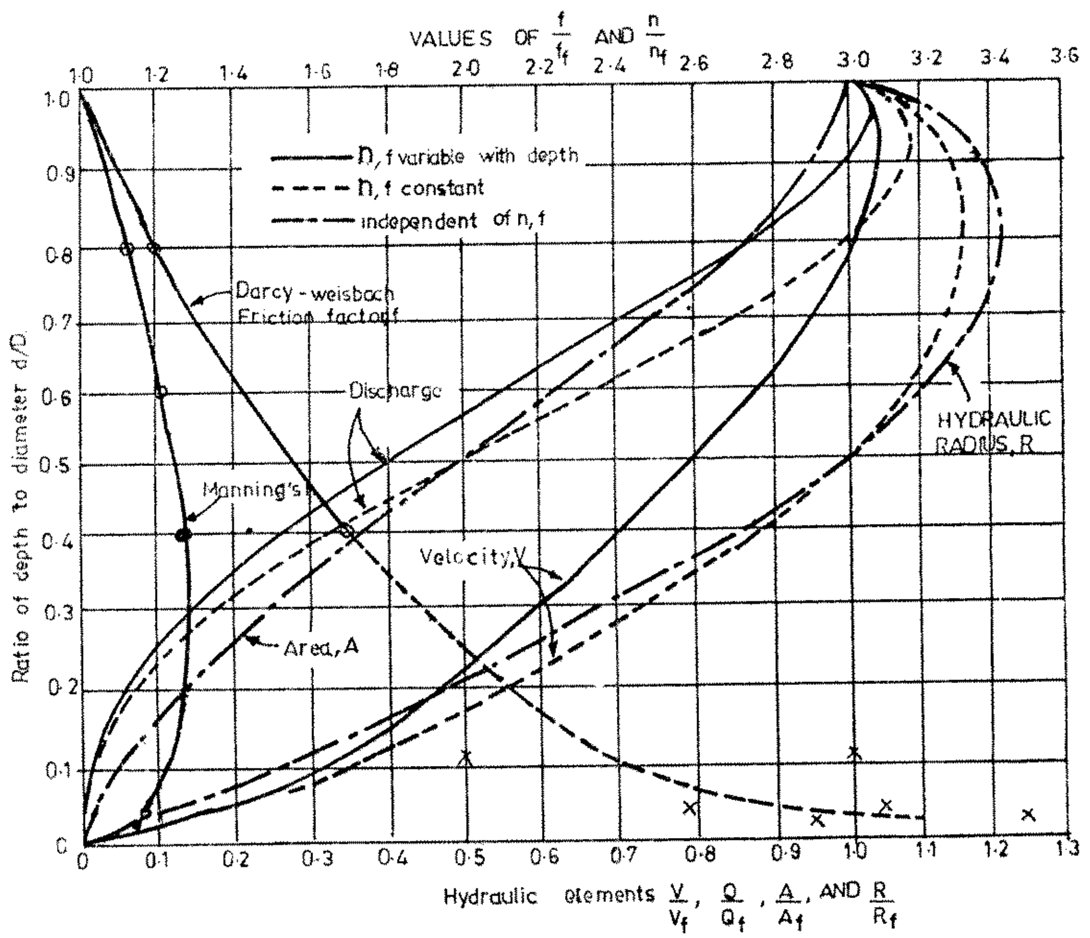


FIG. 3.18 HYDRAULIC-ELEMENTS GRAPH FOR CIRCULAR SEWERS.

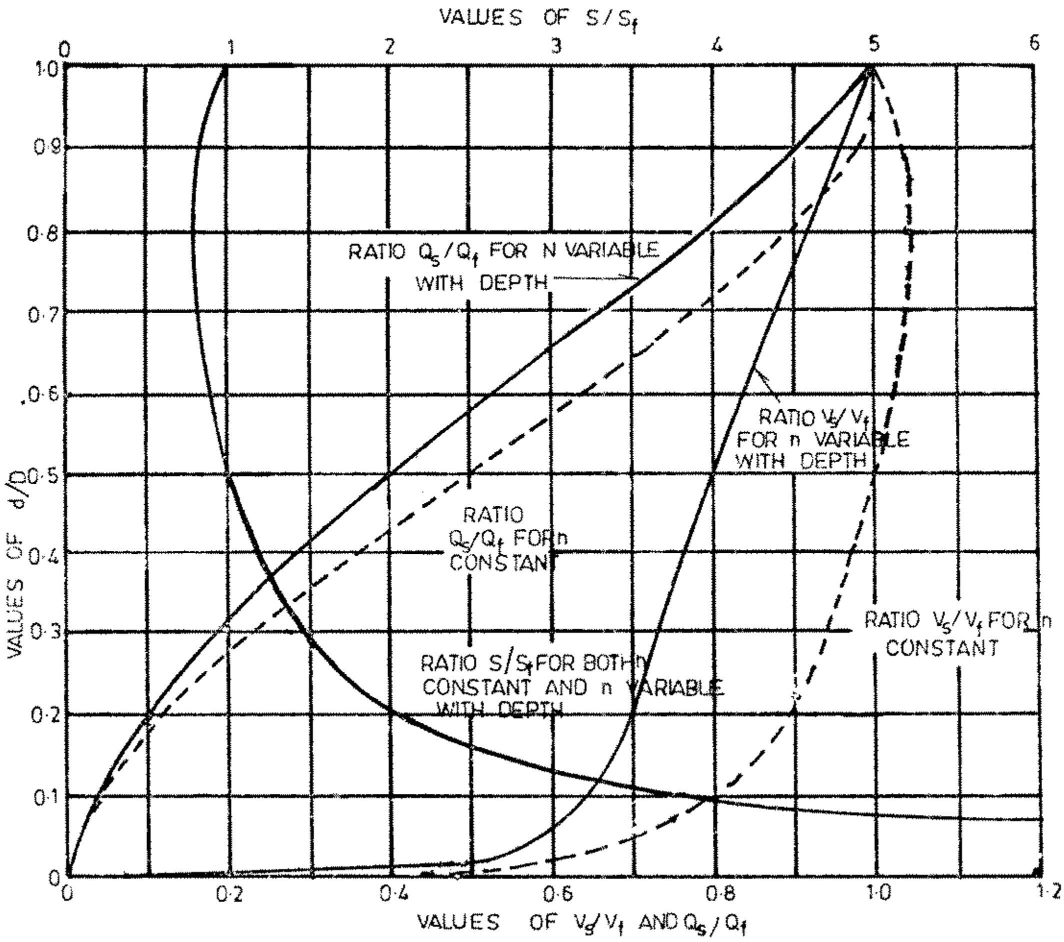


FIG. 3.2: HYDRAULIC ELEMENTS OF CIRCULAR SEWERS THAT POSSESS EQUAL SELF-CLEANSING PROPERTIES AT ALL DEPTHS.

3.4.2.7 Formula for Self Cleansing Velocity

From finding of Shields, Camp derived the formula

$$V = (1 / n)R^{1/6} \{ K_S (S_S - 1) d_p \}^{1/2} \qquad (3.13)$$

In which S_S is specific gravity of particle, d_p is practicle size and K_S is a dimension less constant with a value of about 0.04 to start motion of granular particles and about 0.8 for adequate self cleansing of sewers.

The Shields formula indicates that velocity required to transport material in sewers is only slightly dependent on conduit shape and depth of flow but mainly dependent on the particle size and specific weight. A velocity of 0.60 mps would be required to transport sand particle of 0.09mm with a specific gravity of 2.65. Hence a minimum,velocity of 0.8 mps at design peakflow and 0.6 mps for present peak flow is recommended in the sanitary sewers.

TABLE 3.6
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	0.712
0.6	1.072	0.671	1.21	0.890	0.557
0.5	1.000	0.500	1.24	0.810	0.405
0.4	0.902	0.337	1.27	0.713	0.266
0.3	0.776	0.196	1.28	0.605	0.153
0.2	0.615	0.088	1.27	0.486	0.070
0.1	0.401	0.021	1.22	0.329	0.017

Where,

D = Full Depth of flow (internal dia)
V = Velocity at full depth
n = Manning's coefficient at full depth
Q = Discharge at full depth

d = Actual depth of flow
v = Velocity at depth 'd'
n_d = Manning's coefficient at depth 'd'
q = Discharge at depth 'd'

3.4.3 Velocities

The flow in sewers varies widely from hour to hour and also seasonally, but for purpose of hydraulic design, it is estimated peakflow that is adopted. However it is to be ensured that a minimum velocity is maintained in the sewers even during minimum flow conditions. At the same time the velocity should not be excessive to cause erosion.

3.4.3.1 Velocity At Minimum Flow

It is necessary to size the sewer to have adequate capacity for the peakflow to be achieved at the end of design periods, so as to avoid steeper gradients and deeper excavations. It is desirable to design sewers for higher velocities wherever possible. This is done on the assumption that although silting might occur at minimum flow, the silt would be flushed out during the peak flows. However the problem of silting may have to be faced in the early years particularly for smaller sewers which are designed to flow part full at the end of design period, where the depth of flow during early years is only a small fraction of the full depth. Similarly upper reaches of laterals pose a problem as they flow only partly full even at the ultimate design flow, because of necessity of adopting the prescribed minimum size of sewer. In such situations flushing arrangements may be provided in the initial years.

In the design of sanitary sewer an attempt should be made to obtain adequate scouring velocities at the average or at least at the maximum flow at the beginning of the design period. It has been shown that for sewers running partially full, for a given flow and slope, velocity is little influenced by pipe diameter. It is, therefore, recommended that for present peak flows upto 30 lps, the slopes given in Table 3.7 may be adopted, which would ensure a minimum velocity of 0.60 mps in the early years.

TABLE 3.7
RECOMMENDED SLOPES FOR MINIMUM VELOCITY

Present peak flow in lps	Slope per 1,000
2	6.0
3	4.0
5	3.1
10	2.0
15	1.3
20	1.2
30	1.0

After arriving at slopes for present peak flows, the pipe size should be decided on the basis of ultimate design peak flow and the permissible depth of flow. The minimum diameter for a public sewer may be 150mm. However, the minimum size in hilly areas, where extreme slopes are prevalent, may be 100mm.

3.4.3.2 Erosion and Maximum Velocity

Erosion of sewers is caused by sand and other gritty material in the sewer and also by excessive velocity. Velocity in a sewer is recommended not to exceed 3.0 mps.

3.4.4 Sewer Transitions

3.4.4.1 Non Uniform Flow

For uniform flow in sewers the slope of the energy and hydraulic grade lines are same as the slope of the invert and the depth of flow will adjust to produce a velocity in proportion with the frictional losses.

In non uniform flow, the energy and hydraulic grade lines are not parallel. Flow in sewers is not uniform in all reaches. There will be regions of uniform and non uniform flow. For longer sewers, it is a good practice to plot the hydraulic profile for various reaches. Profile calculations have to begin at a point where depth and velocity are known. In many cases the hydraulic profile can be calculated from a control section where total energy above the invert is a minimum for a given discharge or the rate of flow is maximum for a given total energy. This is known as critical flow or flow at critical depth, where Froude's number F is equal to unity. Froude's Number is defined as

$$F = \frac{V}{\sqrt{gd_m}} \qquad (3.14)$$

where d_m = hydraulic mean depth

If $F^{*} < 1$, the flow is subcritical

and if $F > 1$, the flow is supercritical

For arriving at the profile, the analysis begins at control point i.e. where F = 1 and proceeds upstream when upstream flow is subcritical and proceeds downstream when downstream flow is supercritical.

3.4.4.2 Specific Energy

For a given section and dishcarge the specific energy head is a function of depth of flow only. If the depth of flow is plotted against specific energy, a specific energy curve is obtained, (fig.3.3) which shows that for all flows except critical flow there are two possible alternate stages or depths at which flow may occur for any value of specific energy head and discharge, depending on channel slope, friction and location of control section.

Where a flow passes from a subcritical stage on a gentle sloping channel to a supercritical stage in a steeply sloping channel it must pass a control section. The control section is located in the vicinity of break in grade and critical flow occurs there. Fig.3.4 shows non uniform flow hydraulic profile. The upstream slope which is less than the critical slope is called subcritical slope or a mild slope. The downstream slope which is greater than the critical slope is called a supercritical slope or a steep slope.

3.4.4.3 Hydraulic Jump

Hydraulic Jump is a phenomenon where a flow in a channel abruptly changes from supercritical flow at a shallow depth to subcritical flow at a greater depth. For a flow from a steep to a mild slope,the hydraulic jump occurs which results in a loss of head. The hydraulic jump may be evolved as a device for dissipation of energy such as where a steep sewer enters a large sewer at a junction. The most important consideration is the location of jump. Fig.3.5 depicts the energy conditions to show that the jump must take place on the mild slope. If the required down stream total energy necessary to transport the flow is greater than that which would result if the jump occurred on the mild slope, the jump must take place on the steep slope. In either case there is a backwater or draw down curve from the jump to the break in grade. The loss of head in hydraulic jump may be calculated by the principle,

$$\frac{d_2}{d_1} = \frac{1}{2}\sqrt{(1+8F_1^2} - 1) \qquad (3.15)$$

$$\Delta H = H_1 - H_2 = \frac{(d_2 - d_1)^3}{4 d_1 d_2}$$

In which d_1 and d_2 are depths before and after jump, F_1 is Froude's Number

upstream of flow,

ΔH is loss of head, H_1, H_2 are specific heads of flow before and after jump.

3.4.4.4 Back Water Curves

Back water or draw down curves occur from abrupt changes in sewer slopes, when there is a tree fall or an obstruction to the flow. It is possible in some cases to make a saving in cost by reducing the size of conduit or lowering the roof, thus possibly avoiding over head structures. Hence it is desirable to know the amount by which the depth is increased at various points along the curve and the distance upstream upto which the back water curve extends. Most frequently encountered curves for mild and steep slopes are given in Fig.3.6.

The following formula is used for stepwise calculations of the reach of conduit between cross sections of given depth.

$$\Delta L = \frac{(d + h_v)}{S_e - S_a} \qquad (3.16)$$

ΔL = Portion of reach of conduit

d = depth of flow

h_v = velocity head

S_e = average slope of energy grade line

S_a = slope of invert and

$\Delta(d+h_v)$ is the change of specific energy between cross sections.

An illustrative example for backwater curve is given in Appendix 3.4.

3.4.4.5 Sewer Transitions

Where conduits of different characteristics are connected, sewer transitions occur. The difference may be 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 after wards these changes are distributed through out the length of transition. 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

$$\text{Head Loss} = K\,(V^2/2g) \qquad (3.17)$$

Where $(V^2/2g)$ 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 entrainment may cause backing of flow. Allowance for the head loss that occurs at these transitions has to be made in the design.

Manholes should be located at all such transitions and a drop should be provided where the sewer is intercepted at a higher elevation for streamlining the flow, taking care of the headloss and also 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 should not be made. The crowns of sewers are always kept continuous. In no case, the hydraulic flowline in the large sewers should be higher than the incoming one. To avoid backing up, the crown of the outgoing sewer should not be higher than the crown of incoming sewer.

3.4.4.6 Bends

The head loss in bends is expressed by

$$h_b = k_b\,V^2/2g \qquad (3.18)$$

Where k_b is a bend coefficient which is a function of the ratio of radius of curvature of the bend to the width of conduit, deflection angle, cross section of flow, Reynolds Number and relative roughness.

K_b is approximately equal to 0.4 for 90 degrees and 0.32 for 45 degrees and can be linearly proportioned for other deflection angles.

3.4.4.7 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 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.

3.4.4.8 Vertical Drops and Other Energy Dissipators

In developed areas, it may be sometimes 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 so as to avoid entrapment of air. Air entrapped 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 $h^{1/2} d^{1/3}$ in which h is the height of fall and d is depth of the crest.

Special chutes or steeply inclined sewers are constructed instead of vertical drops. All drops cause release of gasses and maintenance problems and hence should be avoided where possible.

3.4.5 Inverted Syphon

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 should 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 which 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 200mm dia are provided in parallel so that

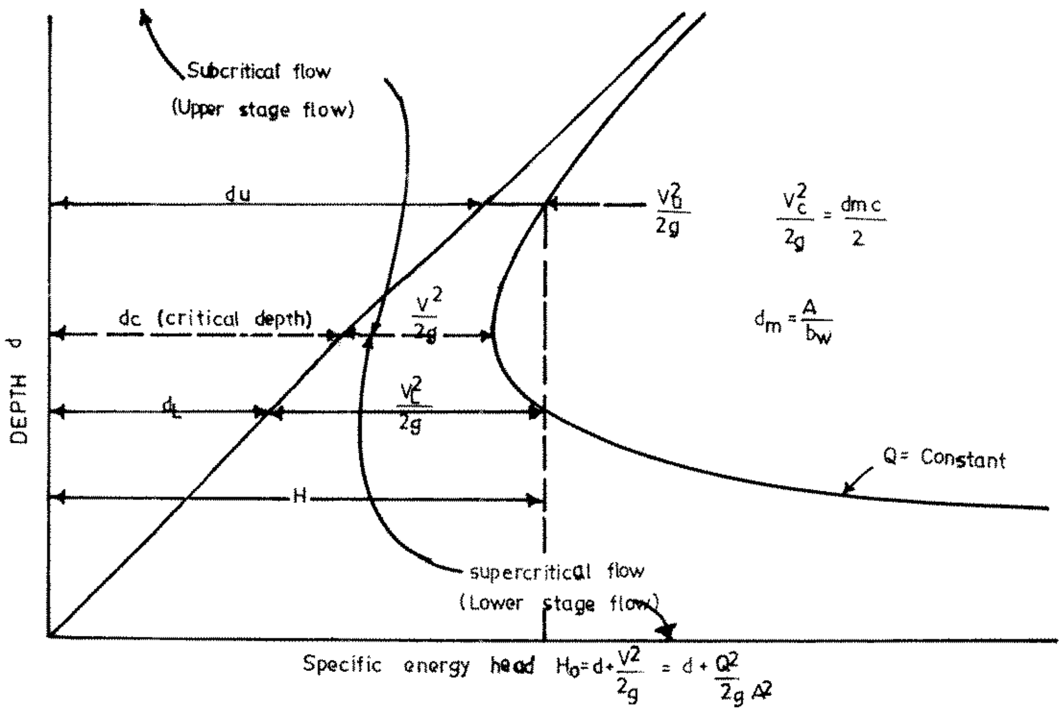
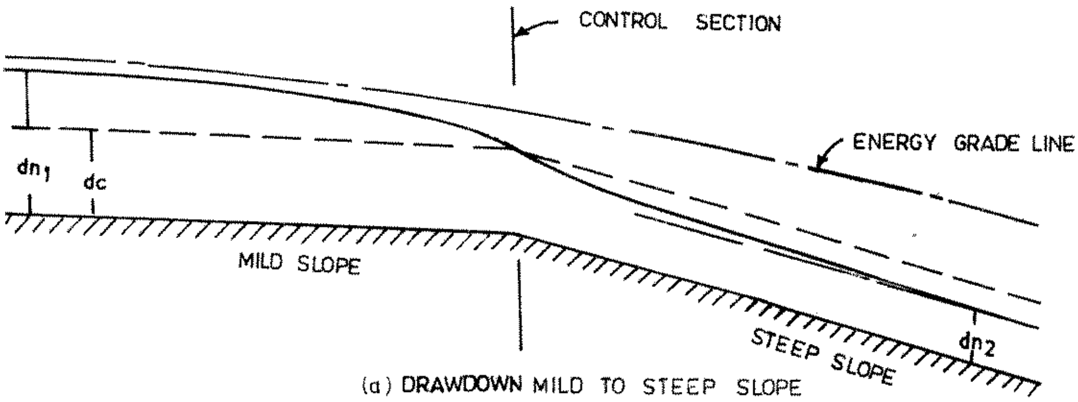
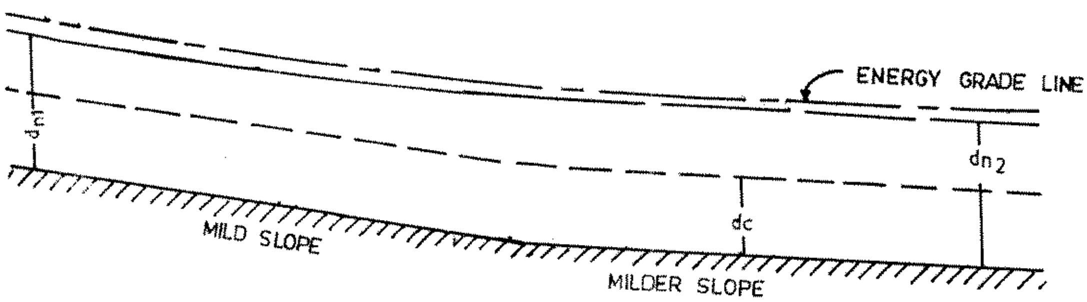


FIG. 3.3: SPECIFIC ENERGY CURVE



(a) DRAWDOWN MILD TO STEEP SLOPE



(b) BACK WATER TO MILDER SLOPE

FIG. 3.4: NON-UNIFORM FLOW HYDRAULIC PROFILES

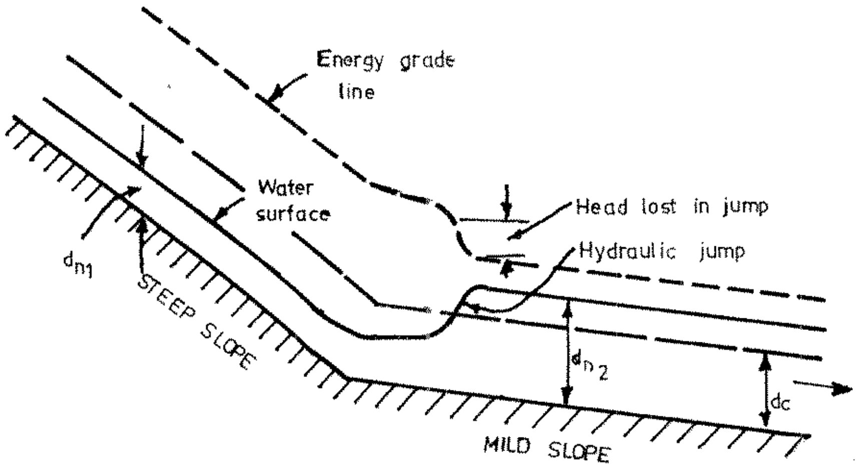


FIG. 3.5: HYDRAULIC JUMP PROFILE