CHAPTER 6

6. Transmission of Water

CHAPTER 6: TRANSMISSION OF WATER

6.1 Introduction

Transmission means the conveyance of water from a source to the water treatment plant (WTP) and thereafter to the distribution system directly or through master balancing reservoir (MBR) or elevated service reservoir (ESR). It includes both raw and clear water transmission. Depending on topography and local conditions, conveyance may be designed for free flow (gravity flow) channels or conduit or pressure conduits. In urban water networks, clear water is normally pumped to an MBR and then conveyed to several ESRs by gravity. This network of pipes that transmits the water without distribution to the consumer is called a *water transmission network*.

There are various types of transmission main systems:

1. Gravity main
2. Pumping main
3. Combined system

6.1.1 Gravity Main

In cases where the source or starting point of the transmission is at a higher elevation and flow in the transmission main occurs from higher potential head to lower potential head, such systems for transmission of water, either open or closed flow is termed as *Gravity System*. A Typical gravity transmission main is shown in Figure 6.1.

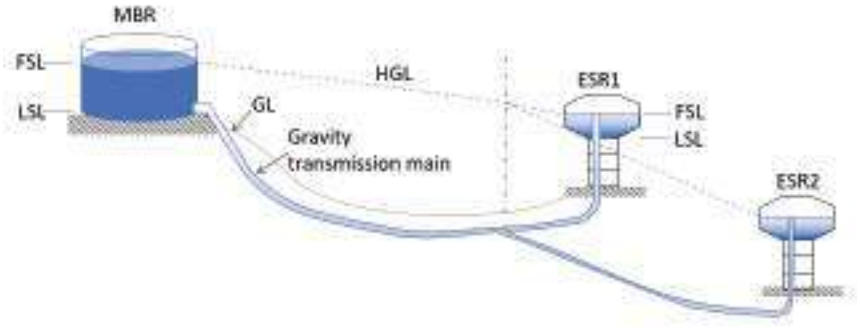


Figure 6.1: Typical Gravity Transmission Main

6.1.2 Pumping Main

When the water has to be transmitted to a higher elevation and starting point of transmission is at a lower elevation, energy/head to the flow has to be provided by an external source. Such a system for transmission of water is termed as *Pumping Main*. A typical pumping main is shown in Figure 6.2. For design of economical diameter please refer to **Annexure 6.1** of Part A of this manual.

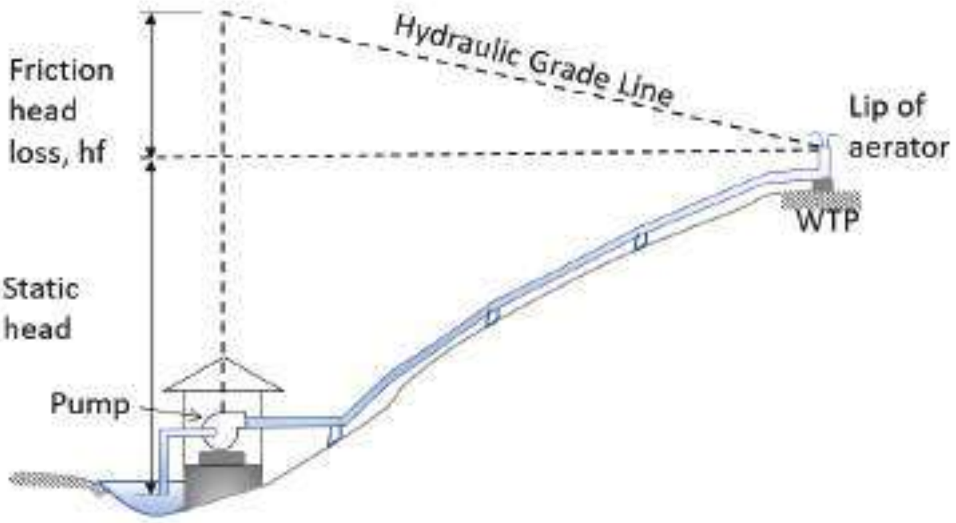


Figure 6.2: A Typical Pumping Main

6.1.3 Combined System

This is the combination of both gravity and pumping mains. Even though sometimes the source/starting point of transmission is at a higher elevation, the advantage of this potential head is not sufficient for the transmission of water. This may be due to friction losses or the presence of a higher elevation enroute to the transmission main. The need arises for providing energy to the system from an external source. Such a system for transmission of water is termed a Combined System.

Another case may be when the water is pumped to a nearby higher or similar elevation from where it can be transmitted by gravity main.

The components for transmission of water account for an appreciable part of the capital outlay and hence, careful consideration of the economics is called for, before deciding on the best mode of conveyance.

6.2 Investigation

(i) **GIS:** For marking the GIS drawing of the transmission main, it is necessary to plot the alignment of the pipeline by adding a path in Google Earth and then saving the path as a Keyhole Markup Language Zipped (KMZ) file which is then converted to the shape file using GIS. GIS mapping is extensively discussed in the advisory on “GIS Mapping of Water Supply & Sewerage infrastructure”, dated April 2020, which is available on the website of MoHUA.

(ii) **Topographical Survey**: Topographic survey has to be carried out to essentially cover details such as alignment/route survey with plan and profile along pipeline alignment, and existing structures with locations of temporary benchmarks. In the case of a temporary benchmark, it is necessary to correlate them, and all drawing are brought on a common datum. The simplest way is to use GIS.

(iii) **Geo-technical Investigations**: Geo-technical investigations that will have to be carried out include bore data, bearing capacity for foundation, rock classification, subsoil water table, quality, etc., will have to be carried out. Soil resistivity will have to be carried out to essentially cover details such as resistivity and basic soil survey by taking trial pits along the pipeline alignments.

(iv) **Resistivity Rating**: This factor is important in deciding which of many protective systems to be adopted for buried pipelines.

6.3 Free Flow and Pressure Conduits

6.3.1 Open Channels/Canals

Canals are generally constructed in their economical trapezoidal cross-section whereas rectangular sections prove economical where rock cutting is involved. They may be lined or unlined depending upon the nature of the ground and available slope. Uniform flow occurs in channels where the dimensions of the cross-section, the slope, and the nature of the surface are the same throughout the length of the channel and when the slope is just equal to that required to overcome the friction and other losses at the velocity at which the water is flowing.

Though they are cheap to construct, they are subject to several drawbacks such as loss of water by infiltration/leakage in the ground, evaporation, pollution, seepage, theft, illegal extraction, and deterioration of water quality by the growth of aquatic plants and/or dumping of waste material. Open channels/canals are not recommended for conveying treated water. However, they may be adopted for conveying raw water. Sometimes diversion channels meant for carrying floodwaters from other catchments are also used to augment the yield from the reservoirs.

6.3.2 Flumes

Flumes are open channels constructed in RCC, either supported on the ground or above ground on RCC pillars to transport water over valleys and other depressions in the path of the conduits or along the deep or rocky side of hilly locations.

6.3.3 Gravity Aqueducts and Tunnels

Aqueducts and tunnels are designed such that they flow three-fourth full at the required capacity of supply in most circumstances. For structural reasons, gravity tunnels are generally horseshoe shaped. Gravity flow tunnels are built to conserve the head and reduce the cost of aqueducts, while traversing uneven terrain. They are usually lined to reduce the head loss and reduce seepage. They may be left unlined when they are constructed through stable rock. Mean velocities, ranging from 0.30 to 0.60 m/s for unlined canals and 1 to 2 m/s for lined canals are maintained to reduce eroding of the channels in due course of time.

6.4 Pressure Aqueducts and Tunnels

Pressure aqueducts are generally constructed in RCC. They are generally circular in cross-section and lined. Pressure tunnels are used in large intake work in lakes, reservoirs, and rivers and as the main feeder of distribution systems. Pressure tunnels are constructed to cross rivers and valleys. Normally, the weight of overburden on the tunnels is relied upon to counterbalance the internal pressure. When there is not enough counterbalance to the internal pressure, steel cylinders or other reinforcing structural arrangement needs to be done to provide necessary strength. They share the advantages of gravity aqueduct and additionally they are not exposed to pollution by seepage waters.

6.5 Pipelines and Force Mains

Force mains/rising mains are pressure conduits or pipelines that carry water from the pumping station to the distribution system or from one level to another higher level. Pipelines are pressure conduits of a circular section that generally follow the profile of the ground surface and are laid below the hydraulic grade line.

The materials used in their manufacture/fabrication are cast iron, mild steel, ductile iron, RCC, pre-stressed cement concrete, polyethylene, asbestos cement (AC) pressure pipes, glass reinforced plastic (GRP), and bar wrapped steel cylinder (BWSC).

Details on pipe materials, their classes, PN ratings, design pressures, factory test pressures, field test pressures, available options for external coating and inside lining/painting with merits and demerits, cathodic protection, methods including impressed current method, hydraulic testing of pipeline in the field (Sectional testing as well as complete pipeline testing), laying of the pipeline, beddings, minimum and maximum cover, river crossing, etc., are described in Chapter 11, i.e., “Pipes and Pipe Appurtenances”.

Further, all valves, such as butterfly valves, sluice valves, air valves, valves of cast steel /SG iron, selection of the diameter of air valve vs pipeline diameter, location of line valves, scour valves, air valves, spacing between air valves, air valves with vertical pipe, valves required to be used above 160 m working pressure or 240 m design pressure, are described in Chapter 11, i.e., “Pipes and Pipe Appurtenances”.

6.5.1 Head Loss in Pipes

When a real fluid flows through a pipe, a part of the total energy is utilised in maintaining the flow. This energy is represented in terms of head of water and when it is utilised, it is termed as head loss. The major head loss in the pipe is due to friction and is termed as frictional head loss. There are several minor losses, which are caused due to changes in the magnitude, direction, or distribution of the velocity of flow.

Using the energy principle, Darcy-Weisbach derived a formula to calculate the head loss. This formula requires trial and error or iterative procedure when used in the analysis and design of water distribution networks. To avoid difficulty in using Darcy-Weisbach’s formula, several empirical formulae were developed. However, Hazen-Williams’ formula for pressure conduits and Manning's formula for free flow conduits have been popularly used.

6.5.1.1 Darcy-Weisbach's Formula

Darcy-Weisbach suggested a dimensionless (dimensionally homogeneous) equation for pipeline problems:

$$h = \frac{fLV^2}{2gD} \qquad (6.1)$$

Where, h = Head loss due to friction over length in metres; f = Dimensionless factor; g = Acceleration due to gravity in m/s^2; V = Velocity in m/s; L = Length in metres; D = Diameter in metres

The Colebrook-White formula can be used for calculation of friction factor, f:

$$\frac{1}{\sqrt{f}} = -2log\left[\left(\frac{k}{3.7D}\right) + \frac{2.51}{Re\sqrt{f}}\right] \qquad (6.2)$$

Where, f = Darcy's Friction Factor or Coefficient; R_e= Reynold's Number = (Velocity × diameter)/Viscosity; k = Average height of Roughness projections.

For more details on the Colebrook-White formula, reference may be made to any standard reference book on Fluid Mechanics.

Reference be made to IS: 2951 for calculation of Head Loss due to friction according to Darcy-Weisbach formula.

Recommended design values of roughness projections (k) for pipe materials are shown in Table 6.1.

Table 6.1: Design Values of roughness projections (k)

S. No.	Pipe Material	Value of 'k' mm	
		New	Design
1	Metallic Pipes Unlined - Cast Iron and Ductile Iron	0.15	*
2	Metallic Pipes Lined - Mild Steel	0.06	*
3	Asbestos Cement, Cement Concrete, Cement Mortar or Epoxy lined Steel, CI, and DI pipes	0.035	0.035
4	PVC, GRP HDPE, PVC-O, and other plastic pipes	0.03	0.03

* Reference be made to {IS: 2951 (Part I)} for roughness values of aged metallic pipes.

6.5.1.2 Hazen-Williams Formula

The Hazen-Williams formula is expressed as:

$$V = 0.849C(r^{0.63})(S^{0.54}) \qquad (6.3)$$

Where, V = Average Velocity of flow in m/s; C = Hazen-Williams coefficient; r = Hydraulic mean radius in m; S = Slope of hydraulic grade line (h/L).

For circular conduits of diameter D, the expression for head loss in terms of discharge can be simplified as

$$h = 10.68\left(\frac{Q}{C}\right)^{1.852}\left(\frac{L}{D^{4.87}}\right) \qquad (6.4)$$

Where, L and D are in metres and Q is in cumecs.

6.5.1.3 Manning's Formula

Manning's formula is:

$$V = \left(\frac{1}{n}\right) r^{2/3} S^{1/2} \qquad (6.5)$$

Where, V = velocity of flow in m/s; and n = Manning's coefficient of roughness, r = hydraulic radius (m), S = slope of pipe, m/m)

For a circular conduit of diameter D, the head loss can be written as

$$h = 10.29\ (Q \times n)^2 \left(\frac{L}{D^{16/3}}\right) \qquad (6.6)$$

6.5.1.4 Coefficient of Roughness for Different Pipe Materials

In today's economic climate, it is essential that all water utilities ensure that their resources are invested judiciously and, hence, there is an urgent need to avoid over designing of the pipelines.

The coefficient of roughness depends on Reynolds number (hence on velocity and diameter) and relative roughness (k/D). For Reynolds number greater than 10^7, the friction factor 'f' (and hence the C-value) is relatively independent of diameter and velocity. However, for normal ranges of Reynolds number of 4,000 to 10^6, the friction factor 'f' (and hence the C-value) does depend on diameter, velocity, and relative roughness.

PVC, glass reinforced plastic (GRP), and other plastic pipes are inherently smoother compared to AC pressure pipes, concrete and cement mortar/epoxy lined metallic pipes. Depending on the quality of workmanship during manufacture and the manufacturing process, the asbestos cement, concrete, and cement mortar/epoxy lined metallic pipes tend to be as smooth as PVC, GRP, and other plastic pipes.

The metallic pipes lined with cement mortar or epoxy and concrete pipes behave as smooth pipes and have shown C-values ranging from 140 to 145 depending on diameter and velocity.

With a view to reduce corrosion, increase smoothness, and prolong the life of pipe materials, the metallic pipes are being provided with durable smooth internal linings. Concrete, asbestos cement, and cement mortar/epoxy lined metallic pipes, PVC, GRP, and other plastic pipes may not show any significant reduction in their carrying capacity with age and therefore the design roughness coefficient values (C-values) should not be substantially different from those adopted for new pipes.

However, pipes carrying raw water are susceptible to deposition of silt and the development of organic growth resulting in the reduction of the carrying capacity of such pipes. In case of the build-up of substantial growth/build-up of deposits in such pipes, they can be removed by scraping and pigging the pipelines.

Unlined metallic pipes under several field conditions such as carrying waters having a tendency for incrustation and corrosion, low flow velocity and stagnant water, and alternate wet and dry conditions (resulting from intermittent operations), undergo a substantial reduction in their carrying capacity with age. Therefore, lower 'C' values have been recommended for the design of unlined metallic pipes. As such, the use of unlined metallic pipes should be discouraged.

The values of the Hazen-Williams coefficient 'C' for new conduit materials and the values to be adopted for design purposes are shown in Table 6.2. Design purpose ‘C’ values are the same as that of new pipes or lesser. These have been suggested by considering the deterioration of pipe surface over the design period.

Table 6.2: Hazen-Williams Coefficients

Pipe Materials	Recommended C-Values	
	New Pipes$^{@}$	Design Purpose
Unlined Metallic Pipes		
Cast Iron, Ductile Iron	130	100
Mild Steel	140	100
#Galvanised Iron above 50 mm dia.	120	100
#Galvanised Iron 50 mm dia. and below used for house service connections.	120	55
Centrifugally Lined Metallic Pipes		
Cast Iron, Ductile Iron, and Mild Steel Pipes lined with cement mortar or Epoxy/Polyurethane/three-Layer Polyethylene		
Up to 1,200 mm dia.	140	140
Above 1,200 mm dia.	145	145
Projection Method Cement Mortar Lined Metallic Pipes		
Cast Iron, Ductile Iron, and Mild Steel Pipes	130*	110**
Non-Metallic Pipes		
RCC Spun concrete, Pre-stressed Concrete, Bar Wrapped Cement Concrete Pipe Up to 1,200 mm dia.	140	140
RCC Spun concrete, Pre-stressed Concrete, Bar Wrapped Cement Concrete Pipe Above 1,200 mm dia.	145	145
PVC, GRP, and other plastic pipes like MDPE, HDPE***, PVC-O, PVC	150	145
Asbestos Cement pressure pipes	150	140

@ The C-values for new pipes included in Table 6.2 are for determining the acceptability of the surface finish of new pipelines. The user agency may specify that a flow test may be conducted for determining the C-values of laid pipelines.

 # The quality of galvanising should be in accordance with the relevant standards to ensure resistance to corrosion throughout its design life.

**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.*

*** In the absence of specific data, this value is recommended. However, in case authentic field data is available, higher values up to 130 may be adopted.*

**** For calculation of flow rates using the Hazen Williams equation, a constant 'C' of 140 (for dia 75 mm and less) and 150 (for dia greater than 75 mm) may be used. (IS 7634 Part-1-1975)*

The coefficient of roughness for use in Manning's formula for different materials as presented in Table 6.3 may be adopted generally for design purposes unless local experimental results or other considerations warrant the adoption of any other lower value for the coefficient. For general design purposes, however, the value for all sizes may be taken as 0.013 for plastic pipes and 0.015 for other pipes.

Table 6.3: Manning's Coefficient of Roughness

Type of lining	Condition	n
Glazed coating of enamel Timber	In perfect order	0.01
	(a) Plane boards carefully laid	0.014
	(b) Plane Boards inferior workmanship or aged,	0.016
	(c) Non-plane boards carefully laid	0.016
	(d) Non-plane boards inferior workmanship or aged	0.018
Masonry	(a) Neat cement plaster	0.013
	(b) Sand and cement plaster	0.015
	(c) Concrete, Steel trowelled	0.014
	(d) Concrete, wood trowelled	0.015
	(e) Brick in good condition	0.015
	(f) Brick in rough condition	0.017
	(g) Masonry in bad condition	0.020
Stonework	(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	0.02
	(b) In ordinary condition	0.025
	(c) With stones and weeds	0.03
	(d) In poor condition	0.035
	(e) Partially obstructed with debris or weeds	0.05
Steel, BWSC, PSC	(a) Welded	0.013
	(b) Riveted	0.017
	(c) Slightly tuberculated	0.02
	(d) Cement Mortar lined	0.011
Cast Iron and Ductile Iron	(a) Unlined	0.013
	(b) Cement Mortar lined	0.011
Unlined metallic pipes		0.015
Plastic (smooth)/ MDPE/ HDPE/PVC		0.011
Asbestos Cement		0.012
Glass Fibre Reinforced		0.010

The friction factor values in practice for commonly used pipe materials are given in Table 6.4.

Table 6.4: Recommended Friction Factors* in Darcy-Weisbach Formula

S. No	Pipe Material	Diameter(mm)		Friction Factor	
		From	To	New	For Design Period of 30 years
1.	R.C.C.	100	2000	0.01 to 0.02	0.01 to 0.02
2.	A.C.	50	1000		
3.	HDPE/MPDE	20	1200		
4.	PVC - U	20	630		
5.	PVC - O	63	1200		
6.	PVC - C	15	150		
7.	Stoneware	100	600		
8.	C.I. (for corrosive waters)	100	1500		0.053 to 0.03
9.	C.I. (for non-corrosive waters)	100	1500		0.034 to 0.07
10.	Cement Mortar or Epoxy Lined metallic pipes (Cast Iron, Ductile Iron, Steel)	100	2000		0.01 to 0.02
11.	G.I.	15	150	0.014 to 0.03	0.315 to 0.06
12.	PSC	300	2600	0.01 to 0.02	0.01 to 0.02
13	BWSC	250	1900		

* Values of *f* can also be considered from the Moody's diagram. Reference be made to IS: 2951 for calculation of head loss due to friction according to Darcy-Weisbach formula.

6.5.2 Reduction in Carrying Capacity of Pipes with Age

The carrying capacity of the pipeline depends on the diameter and the Hazen-Williams C-value, which is proportional to the smoothness of the interior surface of the pipe. The higher the C-factor, the smoother the pipe, the greater the carrying capacity, and the smaller the friction or energy losses from water flowing in the pipe. The water carrying capacity of pipes decreases with age due to incrustations (deposition of solids). In effect, the diameter of the pipe and the Hazen-Williams C-value get reduced. The reduction in diameter and C-value causes increase in frictional loss and is reflected in the gradual reduction in carrying capacity of the pipeline and reduction in tail end pressures. So, it can be said that the loss in carrying capacity is caused by: (1) a decrease in the cross-section due to the accumulation of deposits on the interior of the pipes, and (2) an increase in the roughness.

6.5.2.1 Discussion on Various Formulae for Estimation of Frictional Resistance

(i) The Darcy-Weisbach formula is dimensionally consistent. However, its use for the estimation of velocity/discharge during the analysis of the network, or diameter in the design of the network is tedious. As the f value cannot be calculated if velocity or diameter are not known, a repetitive method is required. Initially, f is assumed, and the unknown velocity/ discharge/ diameter as the case may be is calculated. Then, the calculated value of velocity/ discharge/ diameter f is obtained using the Colebrook-White formula. If the obtained value is found to be the same, the process is terminated, else the obtained value is considered, and the process is repeated.

(ii) The Hazen-Williams formula is derived for a hydraulic mean radius of 0.3 m and friction slope of 1/1000. However, the formula is used for all ranges of diameter and friction slopes. The formula is dimensionally inconsistent, and the Hazen-Williams C can be considered to have the dimension of $L^{0.37}T^{-1}$, and therefore is dependent on velocity, diameter, and other parameters. However, the Hazen-Williams coefficient C is usually considered independent of pipe diameter, the velocity of flow, and viscosity.

While the DW equation can be used to any Newtonian fluid, the HW formula was created specifically for water. The network's flow is typically turbulent, hence the HW does not deal with laminar flows. It goes without saying that there is virtually no head loss at that low velocity. The answers of the HW and DW equations coincide for a certain Reynolds number. The outcomes somewhat deviate as one goes from that value. The impact is particularly noticeable on rough pipes. However, for smooth pipes, the changes are typically negligible. In cases when pipes with a diameter of 1800 mm or more have exceptionally high Reynolds numbers, it can be necessary to lower the C-factor. The viscosity impact of temperature cannot be readily adjusted. Despite all of these distinctions, they are negligible for ordinary water and sewer operations. For over a century, engineers have been designing millions of kilometres of pipes using the HW equation, and those pipes are still in operation today. It is possible to calibrate models created using the HW equation to match actual piping systems.

(iii) If there is a choice for use of pipe friction formulae, Darcy-Weisbach which yields accurate results can be preferred over the Hazen-Williams (HW) formula. However, no other formula for head loss in pressurised pipe flow conditions should be used.

(iv) Manning’s formula is recommended for flow under atmospheric pressure such as in open channels, and partially filled pipes.

6.5.2.2 Method of Determining Value of ‘C’ for Existing Pipes at Site

Commercial pipes are available in different lengths for different pipe materials. The C-values of individual pipes can be determined in the lab. However, this may not give a correct representation of the C-value of pipes in the field, where pipes are joined in series from one node to the other node. These joints greatly affect the C-value of pipe and therefore, it is sometimes desirable to determine the C-value at the site. The following method can be adopted.

Choose a pipe of the required size of any material for which C-value is required (preferably 100 mm flanged pipe for ease in transportation), transport at a wash water outlet of the existing water supply system, connect with wash water sluice valve flange, tighten the flange of pipe putting rubber insertion between sluice valve flange and pipe flange with nuts and bolts to avoid any leakage. Lay over ground this 100 mm flange pipe at least 105 m in length. Put distinguishable marks 100 m apart on the pipe. The inverted water manometer is accurate and gives a difference of heads up to 1 mm. Hence, it is installed at two marked points 100 m apart on the pipe. Fit ultrasonic flowmeter in between the marks (preferably in the middle). Now, open the wash water valve of the existing water supply to permit water flow. Let the water flow for 5 to 10 minutes and then take at least 10 readings of heads in the manometer at both the marked points and flow rates. Find the density of water by hydrometer by taking five samples of water collected from the outlet of the laid pipe and take five readings. By averaging all the readings, let the following average readings be obtained.

- Average Pressure (first mark (P_1))
- Average Pressure (second mark 100m apart (P_2))
- Average Discharge (flow rate Q)

- Average Density of water (ρ)
- Length of pipe (L)
- Diameter of pipe (D)
- Acceleration due to gravity (g)
- Now, loss of pressure in length 100 m = P_1 - P_2 = P
- Loss of head (h) = P/ρg

Hazen-Williams' formula as in Eq. (6.4) can be used to obtain the C-value of pipe.

Change in 'C' with age can also be determined analytically using the relation given by Sharp and Walski (1988):

$$C = 18.0 - 37.2\, log\left(\frac{\varepsilon_0 + at}{D}\right)$$

Where:

ε_0 = roughness height when pipe was new (t=0) (mm)
a = rate of change in roughness height (mm/year)
t = age of pipe (years)
D = diameter (mm)

The corrosivity of the water causing change in roughness height is related using the Langelier Index, shown in Table 6.5.

Table 6.5: Correlation between Langelier Index and the Roughness Growth Rate

Description	a (mm/year)	Langelier Index
Slight attack	0.025	0.0
Moderate attack	0.076	-1.3
Appreciable attack	0.25	-2.6
Severe attack	0.76	-3.9

The relationship between C and age is related to the base 10 log of the roughness height and diameter.

6.5.3: Minor head loss due to Specials and Appurtenances

Pipeline transitions and appurtenances add to the head loss, which is expressed either in terms of velocity head as

$$h_m = \frac{KV^2}{2g} \tag{6.7}$$

Where V is the average velocity before the minor loss element, and K is the minor loss coefficient that remains practically constant for high Reynolds' number.

The values of K to be adopted for some typical fittings are given in Table 6.6. Hydraulic tables or standard textbooks and reference books or a manufacturer's catalogue can be used for other special fittings.

Table 6.6: K-Values for Different Fittings/Valves

Type of Fittings	Value of K
Sudden contractions/expansion	0.3*- 0.5
Concentric/Eccentric reducer and enlarger	0.15-0.25
Bellmouth	0.1

Type of Fittings	Value of K
Entrance shape well rounded	0.5
Elbow/Bend 90°	0.5-1.0#
45°	0.4-0.75#
22.5°	0.25-0.50#
Tee 90° take-off	1.5
Radial tee	0.8
30/45 degrees tee	1.0
Straight run	0.3
Coupling/Flange adapter/Dismantling joint	0.3
Gate valve/Sluice valve/Knife gate valve (in fully open condition)	0.3-0.4
Globe	10.0
Angle	5.0
Swing check valve/non-return valve/Reflux valve/Dual plate check valve	2.5
Butterfly valve	0.4
Venturi Meter	0.3
Orifice	1.0
Magnetic/Ultrasonic flowmeter	0.1
Discharge head elbow(bend)/Subsurface delivery tee for VT pump	0.5
Foot valve	2.0
Strainer	1.5

* *Varying with area ratios.*

Lowest values are for long radius elbows and highest values are for short radius elbows.

The minor losses in pipes can also be considered through the equivalent length of straight pipe that can be added to the length of the pipe. The equivalent length values of pipe for different sizes of various fittings with K=1 is given in Table 6.7.

Table 6.7: Equivalent Length of Pipe for Different Sizes of Fittings with K = 1

Size in mm	Equivalent length of pipe in metres		Size in mm	Equivalent length of pipe in metres
10	0.3		65	2.4
15	0.6		80	3
20	0.75		90	3.6
25	0.9		100	4.2
32	1.2		125	5.1
40	1.5		150	6
50	2.1			

6.6 Guidelines for Cost-Effective Design of Pipelines

The cost of the transmission and distribution system constitutes a major portion of the project cost. It is desirable to adopt the following guidelines:

(i) In the design of distribution systems, the minimum design velocity should be selected in such a fashion to avoid the deposition at the bottom of the pipe which may result in deterioration of pipe quality. A minimum velocity of 0.4 to 0.6 m/s is recommended to avoid depositions and consequent loss of carrying capacity. However, where inevitable due to minimum pipe diameter criteria or other hydraulic constraints, lower velocities up to 0.3 m/s may be adopted with adequate provision for scouring.

(ii) The maximum flow velocity should not be more than *2.5* m/s for raw water to avoid the abrasion and subsequent scouring in the pipelines due to suspended particles. However, in case of filtered water, as the quantity of solids (which contribute to the abrasion) is negligible, the maximum flow velocity to be adopted shall be 3 m/s.

(iii) For hilly area and branch pipe connecting transmission main to service reservoir:

The maximum velocity for MS/DI pipes with internal mortar lining shall be limited to 4.0 m/s for following two cases:

a) For hilly regions

b) For part of branch pipe connecting transmission main to service reservoir required for dissipation of excess residual head

(iv) In all hydraulic calculations, the actual internal diameter of the pipe shall be considered after accounting for the thickness of the lining, if any, instead of the nominal diameter or outside diameters (OD).

(v) The Head Loss gradient should not exceed 10m/km. The value of head loss gradient can be exceeded in hilly areas, however, the velocity should not exceed the permissible value of 2.5 m/s.

(vi) It is desirable that head loss due to fittings, specials, and other appurtenances are obtained. However, accounting for an individual head loss of each valve and fitting used in transmission mains and water distribution networks (WDN) is not practically possible. Usually, these minor losses are considered as 10% of the frictional losses. In some of the software that are used for the simulation and design of WDNs, there is no provision for a direct increase in friction loss by a certain percentage. Therefore, either the length, flow, or C-value can be modified appropriately. To account for 10% of minor losses, the length of pipes can be increased by 10% or nodal demand can be increased by 5.28%, or the C-value can be reduced by approximately 5%.

6.7 Economical Size of Transmission Main

6.7.1 General Considerations

When the source is separated by a long distance from the area of consumption, the conveyance of the water over the distance involves the provision of a pressure pipeline or a free flow conduit entailing an appreciable capital outlay. Therefore, the most economical arrangement for the conveyance is therefore of importance.

The available fall from the source to the town and the ground profile in between should generally help to decide if a free flow conduit is feasible. Once this is decided, the material of the conduit is to be selected, keeping in view the local costs and the nature of the terrain to be traversed. Even when a fall is available, a pumping or force main independently or in combination with gravity main could also be considered. Optimisation techniques need to be adopted to help decisions.

The diameter (D in m) of a free flow conduit connected between two reservoirs having a head difference of h m to carry a known discharge of Q m^3/s can be simply obtained by using the HW head loss formula (Eq. 6.4). This will result in a non-commercial size that can be changed to the next available higher size.

However, the design of a pumping main requires consideration of both pipe size and pump capacity. A smaller pipe size provides the lower pipe cost, however, results in higher head loss and thereby higher pump capacity and higher energy cost. On the contrary, higher pipe size increases the pipe cost, however, due to lesser head loss, both pump capacity and energy charges are reduced. The optimal diameter is the size that minimises the overall cost of pipeline and pump cost and energy cost. Such a diameter may be theoretical and may not be available. Thus, size from the set of available commercial pipe sizes is chosen to minimise the overall cost and is called as *Economical Diameter*. As different types of costs at different times are involved, the theory of economic analysis is used for the comparison of alternatives.

The most economical size for the conveyance main will be based on a proper analysis of the following factors:

(i) The period of design considered is 30 years or the period of loan repayment if it is greater than the design period for the project and the quantities to be conveyed during different phases of such period.
(ii) The different pipe sizes against different hydraulic slopes/acceptable velocity ranges can be considered for the quantity to be conveyed.
(iii) The different pipe materials which can be used for the purpose and their relative costs as laid in position.
(iv) The duty, capacity, and installed cost of the pump sets required against the corresponding sizes of the pipelines under consideration.
(v) The recurring costs on:

   a. Energy charges for running the pump sets. Escalation in costs per year also needs to be considered. Usually, the escalation/inflation rate per year is 2% less than the rate of interest,
   b. Staff for the operation of the pump sets,
   c. Cost of repairs and renewals of the pump sets,
   d. Cost of miscellaneous consumable stores, and
   e. Cost of replacement of the pump sets installed to meet the immediate requirements, by new sets at an intermediate stage of the design period. The full design period or the repayment period may be 30 years or more while the pump sets are designed to serve a period of 15 years.

6.7.2 Evaluation of Comparable Factors

Every alternative, when analysed on the above lines, can be evaluated in terms of cost figures on a common comparable basis by:

(i) The capital cost of the most suitable pipe material as laid and jointed and ready for service, including the cost of valves and fittings and all ancillaries to the pipeline.

(ii) (a) Capital cost, as installed, of the necessary pump sets corresponding to the pipeline size in (i) above.

(b) The amount which should be invested at present would yield compound interest, the amount necessary to replace the pump sets in (ii)(a) at the end of their useful life with bigger pump sets for once or often to cater to the requirements during the design period or the loan repayment period.

(iii) Energy charges - if the pump sets in (ii)(a) are designed to serve for, say 15 years, the daily pumpage will vary from the initial requirements to the intermediate demand after 15 years. The energy charges will be based on the average of these two daily pumpages, leading to an average annual expenditure on energy charges on such a basis.

The replacing of pumps under (ii)(b) will, likewise, involve annual recurring energy charges for the average of the demands during the subsequent 15 years period for the project design or the loan repayment period whichever is greater.

The two annual recurring costs should be capitalised for inclusion as a part of the present investment. For this purpose, it is necessary to derive:

(a) the amount of the present investment which would yield an annuity for 15 years equal to the annual energy charges on the initial pump sets;
(b) the amount of present investment which would commence to yield, over the subsequent 15 years period, the annual energy charges for the replaced pump sets in (ii)(b);
(c) apart from the energy charges, the other recurring annual charges comprise the cost of operation and maintenance staff, ordinary repairs, and miscellaneous consumable stores.

The present investment which would yield an annuity equal to such annual recurring charges throughout the design period, or loan repayment period (if it exceeds the former), would represent the capitalised cost, for inclusion as part of the total investment now required.

(iv) The addition of the present investment figures as worked out under (i), (ii)(a), (ii)(b), (iii), and (iv) would represent the total capital investment called for in respect of each alternative involving a specific pipeline size and the corresponding pump sets. A comparison of the total investment so required in respect of the several alternatives examined would indicate the most economical pipeline size to be adopted for any project.

(v) In all the above computations, the rate of interest plays an important role and for a proper comparison, it may be taken as the rate demanded for the loan repayment. Also, inflation should be considered and the minimum attractive rate of return, i_r (MAAR) can be obtained by subtracting the inflation rate, i_{in} from the effective interest rate, i_f.

A typical variation of the total cost curve with respect to diameter is shown in Figure 6.3. The curve is a unimodal convex. Therefore, to avoid consideration of all available sizes, few candidate pipe sizes can be selected. This will reduce computational efforts. In case, the economical size is obtained as the lowest or largest from the list of candidate diameters, the process can be repeated by including one of the higher/lower sizes depending on the obtained size. If no higher/lower size is available, the last pipe is the economical size. The number of candidate sizes can be chosen using velocity or hydraulic gradient criteria or using Lea’s approximate formula. Lea suggested that the economical diameter in metre usually lies between 0.97 to 1.22 $\sqrt{Q}$, where Q is the design discharge in the pumping main in m^3/s. Thus, four to five commercial diameters in the above range can be selected as candidate diameters.

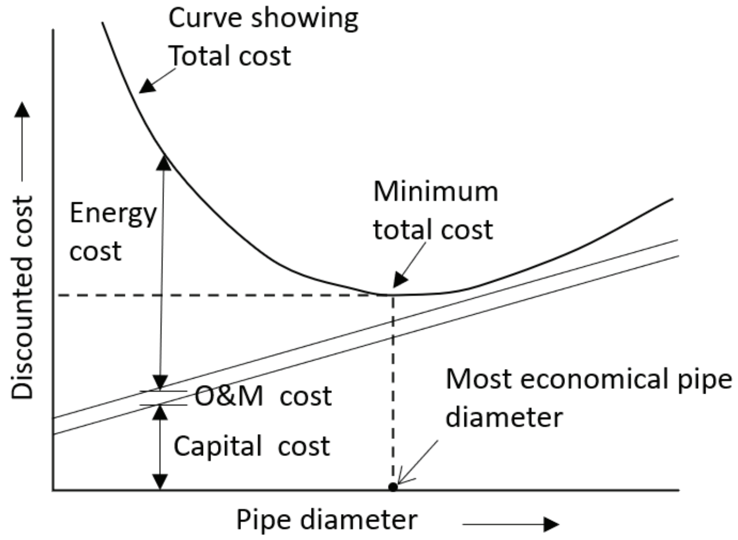


Figure 6.3*:* Variation of Total Cost with Pipe diameter

The stepwise methodology is given and illustrated with an example in **Annexure 6.1**

6.7.3 Scope of Sinking Fund

In the methods of comparison outlined above, any provision for a sinking fund to replace the pipeline or the pump sets at the end of the design or loan repayment period, where needed, has been advisedly not included. It would be tantamount to the present generation paying in advance for the amenities for the next generation, in addition to paying for its own amenities through the design period of 30 years. Such a procedure is neither equitable nor expedient, particularly when local finances are not enough to shoulder the financial commitments even against the initial installations of such projects.

6.7.4 Pipeline Cost under Different Alternatives

There are three independent factors bearing on the problem, viz., the design period of 30 years, the loan repayment period, and the life of the pipeline. There is a particular pipe size for which cost should be minimum, considering its capital and maintenance charge, for the loan repayment period. The size of the pipe will be larger if the period considered is the life of the pipeline and this larger size would appear to be less economical if the period is restricted to the loan repayment period.

The issue, therefore, hinges on which size to choose out of the two in a particular project. Whichever size is adopted, the loan, therefore, has to be repaid within the specified period, long before the pipeline ceases to be of use. For the investor, the pipe size which will cost him/her the minimum is the criterion, pipe costs, and maintenance being considered over the loan repayment period. The other size based on the life of the pipe material would cost him/her an additional financial burden although it may be the cheapest when considered over the life period of the pipeline. For the purpose of finding an economical diameter, adopting the price as per relevant DSR is good enough.

6.7.5 Life of Pipes

'Pipe Life' is the expected 'Design Useful Service Life' (DUSL) for a particular 'Pipe Material'. The life period of the pipeline will depend on several factors which are as follows:

a) Pipe material and thickness

b) Working pressure of the pipeline

c) Workmanship

d) Operation and maintenance

e) Characteristics of water

f) Surrounding environment

6.7.6 Recurring Charges-Design Period vs. Perpetuity

The annual recurring charges for energy and operation and maintenance are perpetual, irrespective of the design period or the life of the pipeline. Their capitalised value is restricted to the design period or the loan repayment period whichever is greater, as it reflects the commitment involved relevant to such period for a proper comparison between alternatives. Otherwise, a possible method may be considered as an initial investment that would yield interest to meet such recurring charges in perpetuity. It is, however, simple and more rational to consider capitalisation of the recurring charges over the design period for the purpose of designing the diameters.

6.7.7 Capitalisation Vs Annuity Methods

In Section 6.7.2(v), the comparison suggested was based on the present capitalised value. Alternatively, the capital installation cost of the pipeline could be converted into an annuity for the design period, or loan repayment period, whichever is greater, in the same way as a loan discharged through annuities. This annuity can then be added on to the other annual recurring charges for a total comparison between the alternatives.

6.7.8 Selection Principles

The above method suggested for evaluation of comparable factors would give a comparative idea of the total capital investment involved whereas the capitalisation vs. annuity methods would indicate the annuities involved as between the alternatives. A better concept is perhaps afforded by the former method, i.e., capitalisation.

The most economical size of a main can be arrived by evaluating the capital and the operation and maintenance cost (capitalised value for design period of 30 years) for different diameters. Mathematical solution is also possible (Annexure 6.1). The objective (cost) function is formulated to ensure desired system performance. Several optimisation techniques are available for minimising the objective function. One of the simpler methods is one in which its (objective function) first partial derivatives with respect to the several decision variables are set equal to zero. The resulting system of equations is solved exactly or approximately and the principal minors of the determinant of second partial derivatives are investigated to ascertain whether a maximum or minimum is involved.

While determining the type of the pipe material to be used, alternative alignments, cost of cross drainage works, cost of valves, specials, and other appurtenances, should all be considered to determine the most economical size for the conveying main.

6.7.9 L-Section

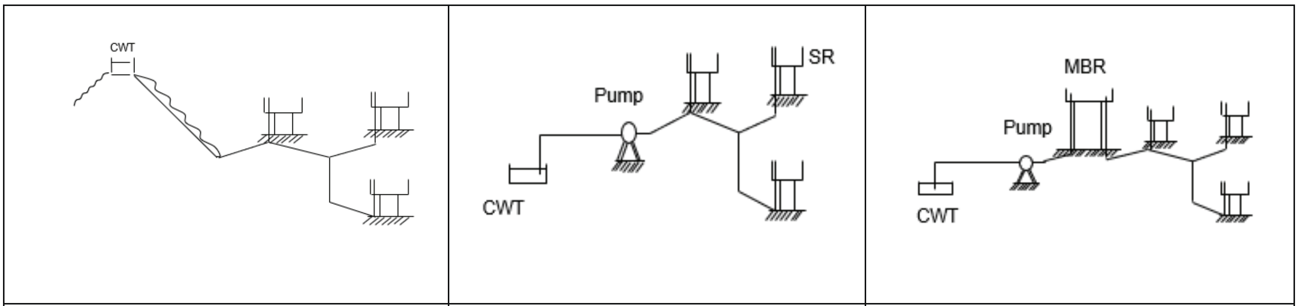
A longitudinal section (L-Section) along the pipeline route must be made to show proper alignment and hydraulic grade after a detailed survey before designing the pipeline, and it is also needed to access the requirements and locations of air valves, scour valves, etc. The L-Section also helps in planning and laying the pipeline and identifying any obstructions and permissions required.

Soil investigation along the alignment to examine the resistivity and corresponding corrosion of soil encountered. Refer to Chapter 11: Pipes and Pipe Appurtenances of Part A Manual.

6.8 Types of Branched Transmission Mains

The economic size design of the pumping main may be said to be a balance between the sizing of the main and the least life cycle cost investment of the system wherein cost of pipes, cost of pump sets, capitalised cost of energy, capitalised cost of operation and maintenance, etc., are considered comparatively for various available sizes of pipes. The pumping main or conveyance main transports water from one location to another location and is not permitted to be tapped between the point of propulsion and the point of reception. However, there could be a direct pumping system feeding to several reservoirs through a network of pipes, or a combined gravity and pumping system in which water from a clear water tank (CWT) at WTP is pumped to an MBR, which in turn supplies to various service reservoirs by gravity. Wherever topology permits, water from the WTP can also be supplied to various reservoirs completely by gravity also.

A typical complete gravity, direct pumping, and combined gravity and pumping system are shown in Figure 6.4 (a), (b) and (c).



(a) CWT - Clear Water Tank at WTP

(b) SR - Service Reservoir

(c)MBR - Master Balancing Reservoir

Figure 6.4: (a) Complete Gravity (b) Direct Pumping (c) Combined Gravity and Pumping

The layout of the transmission main system has great importance on the cost of the network. The layout of a distribution network depends on the existing pattern of streets and highways, existing and planned sub-division of the service area, property right-of-way, possible sites for ground and ESRs, and location and density of demand centres.

Pipes, being lifelines, should be laid along the roads. A minimum spanning tree or shortest path tree from CWT to various ESRs can reduce the cost substantially and should be preferred. Grouping high-level and low-level ESRs in the city should be done, preferably by the use of the GIS technique of the inverse distance weighted (IDW) surface. However, duplication of the pipeline, i.e., parallel pipelines, should be avoided. If necessary, alternatives for layouts can be considered and the one providing the least cost can be selected.

The topography of the service area may be flat or uneven. In an uneven terrain, booster pumps may be necessary for pumping water to high areas within the network. Similarly, it may be necessary to provide pressure-reducing valves for areas with lower elevation to reduce pressure. Check valves (non-return valves) may also be necessary to maintain flow in the selected direction and restrict flow from the opposite direction. The transmission main systems are used for supplying water to various service reservoirs in the city. They are also used in group water supply schemes, in which several villages or a combination of urban towns and villages are supplied from a common source and WTP facilities.

The supply from CWT/MBR to various village/town reservoirs may be direct as shown in Figure 6.5 (a). Such systems may be termed as single level systems. Sometimes, MBR may supply to several zonal balancing reservoirs (ZBRs) which in turn may supply to several village reservoirs (VRs) as shown in Figure 6.5 (b). Such systems may be termed as multi-level systems.

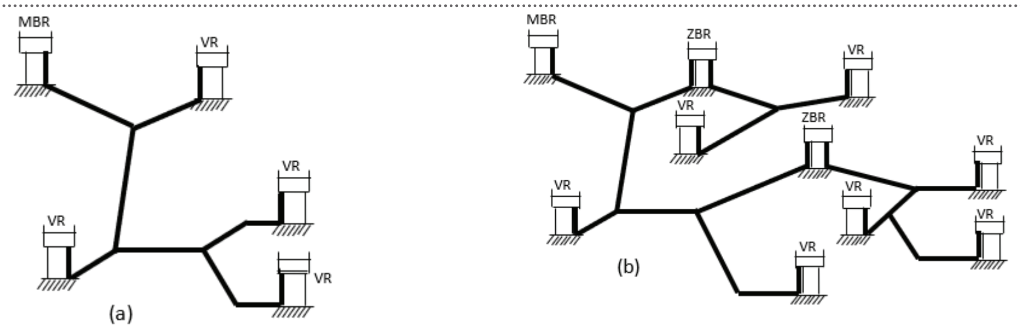


Figure 6.5: Water Transmission System: (a) Single Level; (b) Multi-Level

Note: MBR - Main Balancing Reservoir; ZBR - Zonal Balancing Reservoir; VR - Village Reservoir

6.8.1 Optimisation of Branched Transmission Mains

Several methods for the optimal design of branched networks are available. The methods of the Linear Programming based model and the hydraulic model are discussed here.

(a) Linear Programming (LP) based model: Linear Programming (LP) based model is most useful for the design of branched networks as it provides a global optimal solution considering discrete pipe sizes. The use of Integer Linear Programming (ILP) will avoid the selection of two sizes. BRANCH software, based on LP and JALTANTRA based on ILP, can be used for the optimal design of the transmission main network. Several metaheuristic techniques like Genetic Algorithm, Simulated Annealing, Cross Entropy Optimisation, Particle Swarm Optimisation, etc. have been tested by researchers to obtain an optimal solution and can be used. But presently, some of these software options are costly and the same is still not giving truly optimised solutions. Moreover, this software is non-spatial and hence, difficult to manage on the GIS platform.

Using JalTantra Software: "JalTantra" is a freeware system for the optimal design of branched water distribution networks, developed by CSE IIT Bombay. The user has to log in to the website (https://www.cse.iitb.ac.in/jaltantra) to access the JalTantra. JalTantra can be used for all types of water transmission mains (WTN), i.e., gravity, pumping, and combined pumping and gravity networks. In the case of a combined pumping and gravity WTN, JalTantra allows the sizing of pumping main, pump, ESR, and gravity mains simultaneously instead of considering them separately. JalTantra considers a constant flow of pumping for the design period of 30 years. This is the main limitation of its use in the design of direct pumping and combined pumping and gravity networks. This free software is a Windows format of the earlier BRANCH programme which was working on the DOS system. However, GIS-based operations are not possible on this software, as with most of the other free software on distribution network modelling and design.

The JalTantra software can be used for optimising the diameters of transmission mains. For 24×7 water supply, equalisation of residual pressures at FSL of service tanks is most important. Without equalisation of pressures, there would be an inequitable distribution of water to the service tanks. Thus, operational zones on lower elevations would get more water with excess pressure and those on higher elevations will get less water with less pressure. After making equalisation of residual heads at the FSL of the storage tanks receive water just equal to their design requirement. Hence, without equalisation, the design of the transmission main is incomplete.

Although the JalTantra software works on the Windows operating system, it is non-spatial. Hence, the user has to give data on the lengths of pipes and the elevation of nodes manually. In case a designer wishes to use modelling and simulation through freeware or commercial software, the traditional iterative method of design using GIS can be adopted.

(b) Hydraulic Model: The design can be made using GIS-based hydraulic model. The model can be prepared using freeware or commercially established software. The brief procedure is as below:

MBR, R1 supplies water to the 7 demand nodes (ESR nodes). The steps involved are shown in the flow chart shown in (Figure 6.6), in which N2 to N6, N8 and N9 are the demand nodes (shown in red colour) representing the service tanks, and N7 is the intermittent junction on a ridge with no demand. In the hydraulic model, elevations to be given at junctions N2 to N6 and N8 and N9 ~~N-7~~ are the FSLs of respective ESRs, whereas ground elevations are given to the junctions N-1 and N7, which are intermediate nodes (not demand nodes).

Normally, assumed diameters, lengths (in case of non-GIS), pipe material, lowest supply level (LSL) of MBR and FSL, and ultimate stage demands are fed to the demand nodes as data. After assigning the data, the hydraulic model is run. Required iterations are carried out by changing assumed diameters suitably, by using the above general principles.

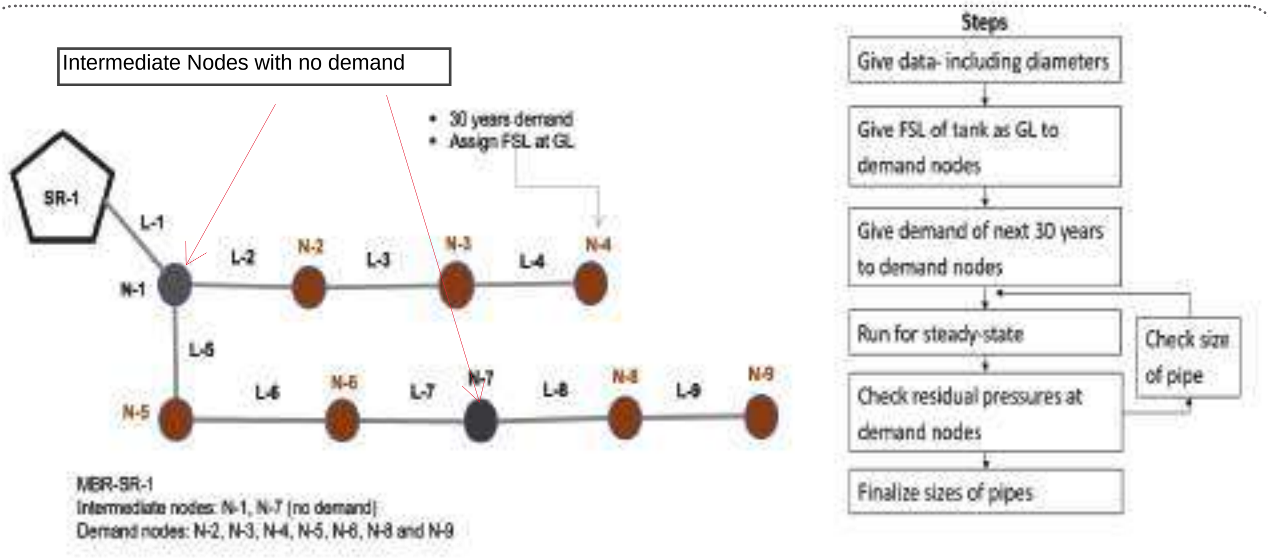


Figure 6.6: Iterative Design of Pipe Diameters of Gravity Transmission Mains

The software analyses the data and computes the residual head at the inlets (FSLs) of each ESR to be served by that MBR. The iterations are carried out till the residual head at FSL of some of the tanks becomes nearer to 3 m.

The iterative procedure for optimisation of diameters of any transmission main in a hydraulic model is shown in Figure 6.7 and is explained below:

1) After running the hydraulic model of the transmission main, we get two tables: (i) pipe table and (ii) junction (node) table. The pipe table contains pipe diameter, velocity, head loss (h_f), and head loss gradient (h_f/km). The junction (node) table contains residual nodal heads. During each iteration of the run of the hydraulic model, both the pipe table and junction tables are kept open so that the pipe diameters, its head loss (h_f) and head loss gradient (m/km) and the residual nodal pressures (m) can be observed simultaneously.

In the pipe table, sort diameters in descending order, and observe values of velocity and head loss, h_f (m/km) in adjoining columns of the junction table.

2) Decrease diameters of the pipes in which velocities are too low and whose diameter is more than 100mm and again run model.

3) Observe the values of velocities in the pipe table. If velocity is less than 1 m/s and h_f (m/km) is also less than 10 m/km and minimum nodal pressure is also more than or equal to residual nodal head as per norm (3 m), the steps are repeated.

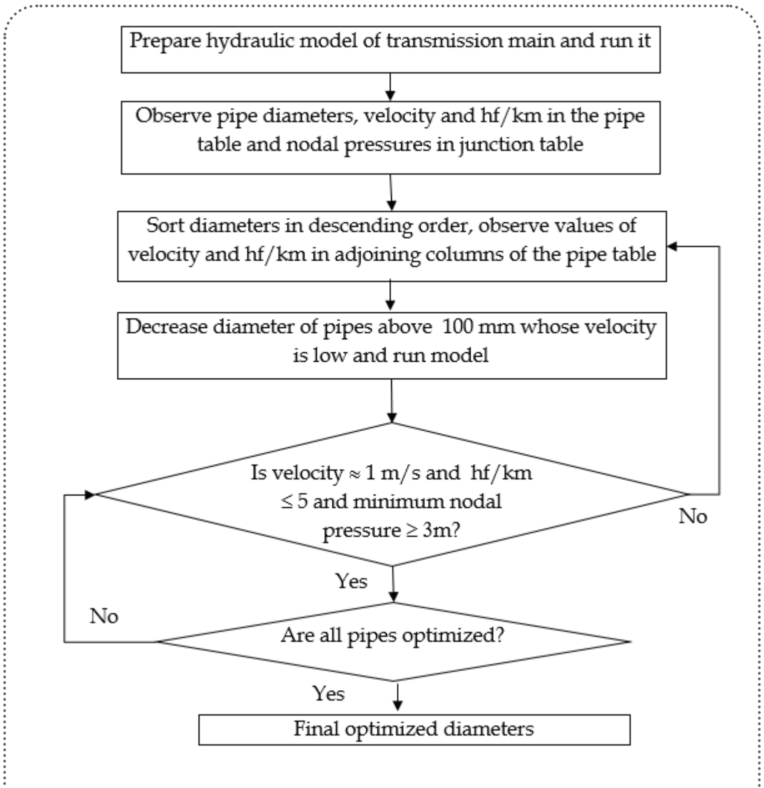


Figure 6.7: Iterative Process

4) The process is repeated for all the pipes whose diameters are more than 100 mm (which is minimum diameter), till we get all optimised diameters.

6.9 Complete Gravity Water Transmission Mains

In a complete gravity network, the supply of water is from MBR to various service reservoirs by gravity as shown in Figure 6.8. The MBR may be located at the ground level as shown in Figure 6.8 or may be elevated as in the case of a multi-level system involving ZBRs. These LSLs of ZBRs are determined considering topography and HGL requirements of reservoirs under respective ZBRs. Thus, the supply level at the source is arrived, which can be considered as the LSL in MBR.

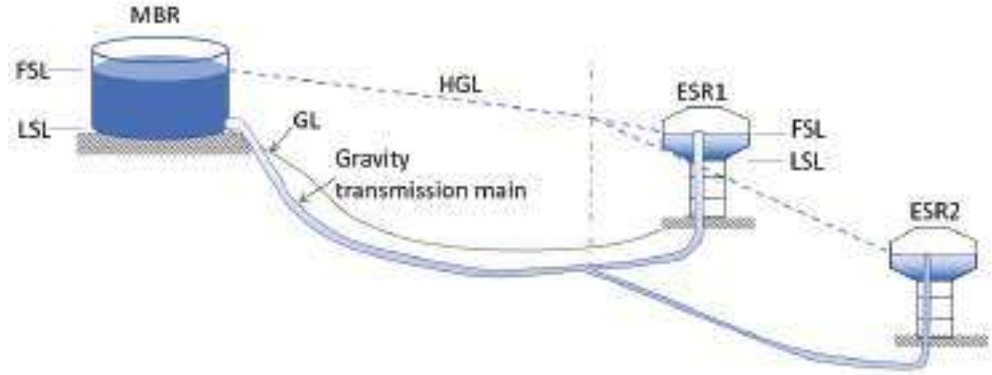


Figure 6.8: A Typical Gravity Transmission Main

6.9.1 General Principles of Design of Gravity Transmission Mains

In general, the following principles are to be adopted in the design of transmission mains by gravity:

(i) After designing optimised boundaries of operational zones of the distribution system, the LSLs of all tanks are known. By adding the necessary side water depth (SWD), we get FSLs of all ESRs. The transmission main shall be designed to give a minimum residual head of 3 m at FSLs of every service tank which is to be fed by the transmission main. The residual head should be as close as possible to 3 m so that the quantity of water supplied to the service tank is nearly equal to the demand of the operational zone that the service tank is serving.

(ii) Grouping high-level and low-level service tanks (ESRs) in the city should be done, preferably by use of the GIS tool of the IDW surface. A case study of the grouping of low and high-level

ESRs is provided in **Annexure 6.2**.

(iii) Lower level group of ESRs should be fed from a low-level MBR and a higher level group of ESRs should be fed from a high-level MBR, through separate transmission networks so that only the needed quantity is pumped to the high-level MBR. This arrangement makes substantial saving in monthly recurring energy bills on account of pumping.

(iv) Several methods for the optimal design of branched networks are available. Linear Programming (LP) based models are most useful for the design of branched network as they provide global optimal solution considering discrete pipe sizes. In the case of Water Transmission Networks (WTN)s, link lengths are more and the residual head on each service reservoir is required to be equalised. The LP in this case not only minimises the cost but also tries to equalise the residual heads at the service reservoirs. Therefore, BRANCH and JALTANTRA software based on LP should be preferred for the optimal design of the transmission main network.

(v) The use of modelling and simulation by free software or commercial software, the traditional iterative method of design can be adopted, which is discussed in Figure 6.7

(vi) Any method based on a single size for each link will produce a higher residual head at each reservoir. Therefore, for WTN where residual head equalisation is a must, the sizes of the branch mains, if possible, should be partly reduced using the moving node method as described in the subsection 6.9.2.

(vii) Criteria for velocity (m/s) and head loss (hf in m/km) are discussed in Section 6.6.

(viii) The diameter of the transmission main on downstream of MBR should not be excessively more and can be little more than that of the inlet diameter of the pumping main (on the upstream side of MBR) feeding the MBR. As the pumping main has a well designed economical diameter, it is used as a guiding factor.

(ix) If assumed diameters after analysis indicate that many ESRs get negative residual head, the MBR level needs to be suitably increased in ca815428815se of new scheme. Thus, LSL of MBR and diameters of transmission main are arrived. Then, the following review should be taken:

(x) For minimising energy cost, it is necessary to lower down LSL of MBR to the extent possible, but this increases the capital cost due to increase in diameters in transmission main.

(xi) From the main network, there is an exclusive branch to feed the ESR at its end, and increasing or decreasing the diameter of that branch does not involve tangible capital expenditure, hence, the diameter of that branch can be increased or decreased to make the network hydraulically and energy cost wise efficient.

(xii) In a large network of transmission main, if only one or two ESRs, that are yet to be constructed, show negative/insufficient residual head, then for such critical ESRs, the following arrangement may be considered:

- Decrease side water depth
- Increase the diameter of branch pipeline to the critical ESR and remember diameter of main lines should not be increased.
- Critically examine the LSL provided for that ESR and decrease it by a meter or so by attempting reduction in head loss in distribution of the relevant OZ/ DMA and by increasing diameter of the feeder main to DMA. Decrease of 1m in LSL of critical ESR leads to decrease of LSL of MBR by 1m, which makes sizable reduction in energy cost as total water needs to be lifted by decreased head of 1m.

- Lower the FSL of these ESRs suitably so that the designed quantity of water from MBR is assured to reach these ESRs and deficiency of nodal heads in the distribution system is redressed as under:
- Provide an online pump on the outlet of such ESRs which will feed its service area; or
- Provide a pump on the pipeline leading to the inlet of pressure deficient DMA served by that ESR.

This arrangement is more economical than increasing the level of MBR and pumping total water to high elevation or increasing the diameter in long lengths of the network. If the height of any of those ESRs needs to be decreased too much, it is better to go for a sump and pump house. From the sump, water could be pumped to the ESR of that operational zone or directly pumped into the distribution system of that OZ.

(xiii) If the main line of the transmission network goes down the slope and again rises, then the ESRs on branches in the lower level will have a high and unrequired residual head. This scenario in the branch line feeding group of ESRs at a lower level is an indicator that the pumping energy is being wasted. On the other hand, while proposing alignment of the main line along a road on a high contour, care should be taken that the top of the pipeline is below HGL by 1 m at least at a critical place. This type of critical place also gives a signal for providing a sump and pumping to downstream ESRs on high locations. It also indicates for providing a ZBR and pumping water to it, and for gravitating water to high-level ESRs. To ascertain this aspect, it is necessary to add nodes showing the elevation of ridge points.

6.9.2 Equalisation of Residual Head

In an ideal design of water transmission networks (WTNs), residual heads at all the ESRs should be the same as the minimum required ones. JalTantra has the capacity to produce such designs. However, because of the topological conditions, minimum pipe diameter conditions, and other inherent conditions in design, residual pressure at all the ESRs may not be observed the same. The performance of the system, when left to itself, would be different from the design one. In practice, the heads more than the minimum required ones increase the pipe discharges until the excess head becomes practically nil. In short, the performance of the system, in actuality, is head-dependent, rather than flow-dependent, as assumed in the design. In order to match the flow-dependent and head-dependent performances of the leading main system, it will be necessary to make the available flow rates equal to the required flow rates at different reservoirs. This can be achieved by dissipating the excess head in the leading mains supplying water to city/village service reservoirs by achieving equal residual head at FSLs of all ESRs. This will make the available flow rates practically the same as the desired ones at all service reservoirs.

The dissipation of the excess head and thereby flow adjustment can be achieved through the provision of pressure-reducing valves. Since they are costly and their fine-tuning to the desired level is difficult, some simple head-dissipating devices can be used. These are: (1) replacement of a part of the branch leading main by a smaller diameter pipe; (2) provision of one or more orifice plates; (3) partial closure of a valve in the branch leading main; or (4) a combination of the three. Since the head dissipation through partial replacement of the existing pipe by a smaller diameter pipe as well as that through the provision of orifice plates alone becomes a permanent solution and does not provide flexibility for easy adjustment in the future, they alone should not be used. For flexibility and fine tuning partial closure of inlet valve is also needed. The solution, therefore, should consist of one of the following measures:

(i) *Partial closure of valve*. When the head to be dissipated is small, a valve provided in the pipe can be partially closed so that the flow can be restricted to match the design flow. Herein, the valve is working as a head-dissipating device. Its adjustment, as recommended by the designer and to be fine-tuned during the trial run, will not be tampered with in the day-to-day operation of the system. Any adjustment that may be necessary in future for changed demands will be made by the central agency.

(ii) *Partial replacement of a branch leading main. W*ith a smaller diameter pipe, this is the best solution. Furthermore, it results ina decrease in cost of the branch leading main. When the head to be dissipated is extremely large and the discharge in the leading main is small, the number of orifice plates in solution 2 is excessive. For such situations, a solution consisting of (a) partial closure of a small diameter valve, (b) one or two orifice plates, and (c) partial replacement of the leading main by a small diameter pipe should be provided. Measure (a) would provide adjustment for fine-tuning during calibration, while measure (b) would help in the adjustment of discharge in the future if measure (a) alone is not sufficient.

(iii) *A combination of partial closure of a small diameter valve and orifice plates*. When the head to be dissipated is large, and the length of branch leading main is small, provision of only partial closure of a valve would not be advisable to dissipate the excess head. Herein, some orifice plates are used in addition to the partially closed valve.

Apart from the above three methods, the Moving Node method (if using a hydraulic model) is most effective.

6.9.3 Moving Node Method

Hydraulic models as well as evolutionary-based design techniques provide designs with a single size for each link and result in higher residual heads. The concept of a single pipe size for each link is understandable for water distribution networks, wherein nodes are closely located. In transmission mains, the distance between the nodes may be several kilometres. Therefore, to save on cost and reduce excess pressure, additional nodes can be generated, and part of the link can be replaced by smaller diameter pipes.

A simple method called as “moving node method” is proposed to achieve these dual objectives of reducing the cost and to equalise the heads. The method works iteratively and stops when residual heads at all the reservoirs are equalised.

From the main network of transmission main, every ESR/GSR has an exclusive branch that serves as an inlet to that ESR. The velocity (m/s) and hf (m/km) in this branch are to be increased by decreasing diameters for dissipating excess residual head. For this purpose, the length of the branch main should be divided (Figure 6.9) into two segments, say L1 and L2 by providing an extra node at the meeting point of (junction) of L1 and L2.

By assigning decreased diameters to the segment connecting the reservoir and by adjusting its length by moving the node at the junction of L1 and L2, the residual head is brought down as close as possible to 3 m. This needs to be repeated for each branch. An increase in velocity up to 4.3 m/s in a small length does not cause any problem as some extra margin is available above the criteria of a minimum 3 m residual head. The design obtained using the moving node method will have two sizes for each branch in the network. The logic of this process in the hydraulic model is shown in Figure 6.10.

The solution may not be exactly the same as obtained by LP-based algorithm but will be close to that and depending on the experience of the designer.

It may be noted that the suggested solution would require a minor adjustment in the field. This fine-tuning can be done during the trial runs. The head-dissipating devices (valves, orifice plates, and smaller diameter pipes, if any should preferably be located on branch lines near the downstream end of the transmission main. This will ensure the hydraulic gradient is above the centreline throughout, thus avoiding the formation of sub-atmospheric pressures in the leading mains.

However, when the head to be dissipated in a long leading main is large, orifice plates and reduced pipe lengths may be provided partly at intermediate places to avoid subjecting the entire leading main to large heads. The head-dissipating valve, however, should be provided at the downstream end. Spacing of at least 100-times the diameter of the leading main between adjacent head-dissipating devices should be used so that normal flow is established between adjacent head-dissipating devices.

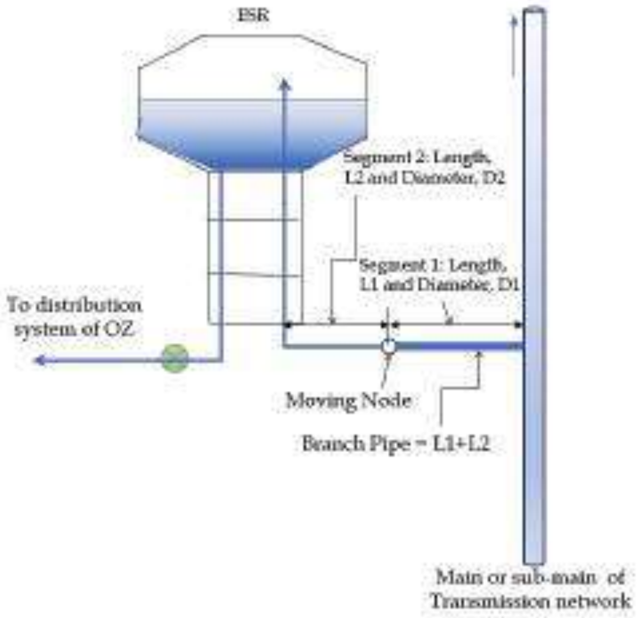


Figure 6.9: Branch Pipe with Two Segments

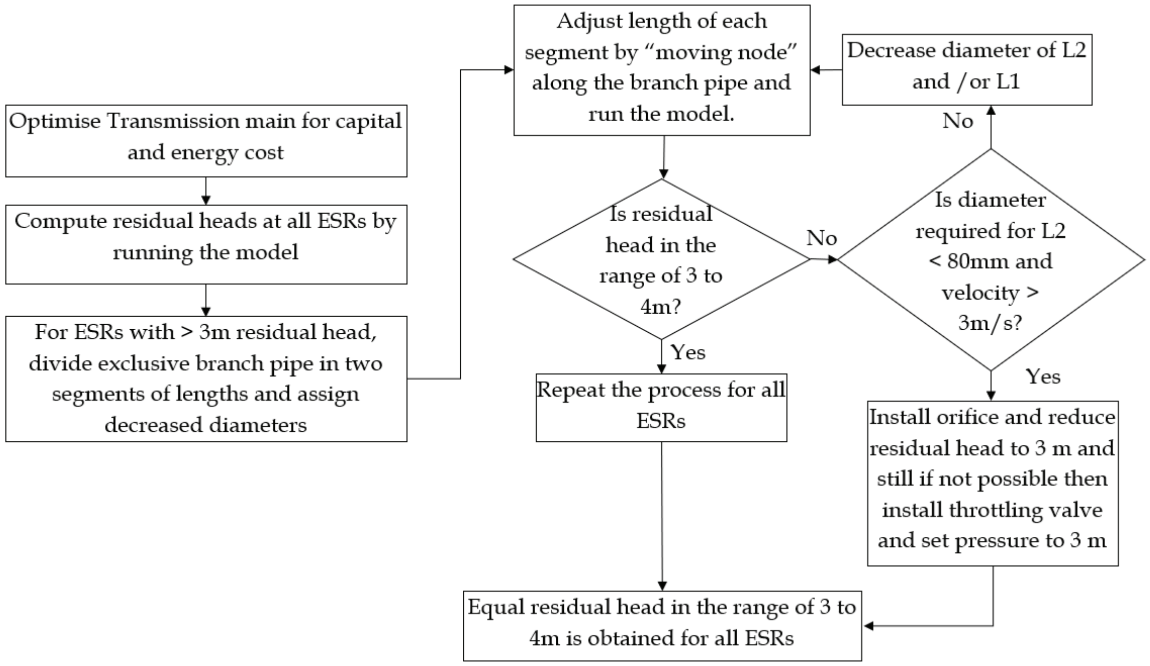


Figure 6.10: Logic of Making Equalisation of Residual Pressure

Designer on drawing, should show a table showing hf/km and velocity in m/s for main stretches of transmission main so that the passing authority can visualise optimisation of cost.

A case study of complete gravity transmission main is presented in **Annexure 6.3**.

6.9.4 Manifold

Sometimes, it is desired to provide head-dissipating device on large diameter main pipelines, especially when larger than the required size is selected to restrict velocity/head loss gradient. This usually happens in hilly regions. Pipes are laid at high slopes and have excessive pressures. The dissipation of head can reduce excess pressure and controls the flow. However, the provision of a large diameter valve increases the cost of the network, and its operation in the field would be difficult. In such cases, a provision of pipe-valve assembly is more useful. Few such pipe-valve assemblies are installed in a Rural Regional Water Supply Scheme (RRWSS) supplying water to 2 towns, 'Daryapur' and 'Anjangaon', and 156 villages in Amravati District. The source of water for this RRWSS is Shahnoor Dam.

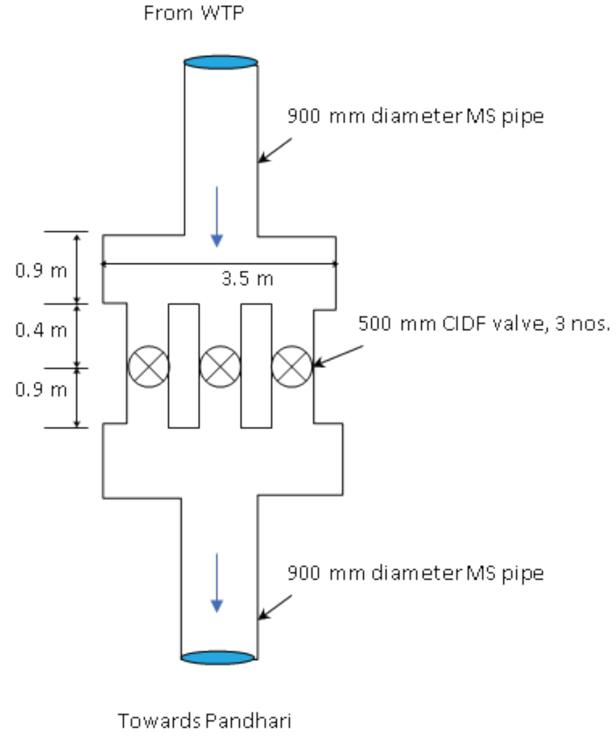


Figure 6.11: Pipe-Valve Assembly near Phandari Phata

Intake works are located on the canal from Shahnoor Dam. The scheme consists of the supply of water from the sump at WTP to 11 MBRs which in turn supplies to 103 villages ESRs. The flow from the intake to village reservoirs is completely by gravity. A typical pipe-valve assembly provided near 'Phandari Phata' on a 900 mm diameter pipe is shown in Figure 6.11. The assembly consists of three parallel pipes of 500 mm diameter pipes connected between two barrels of 900 mm size. One valve of 500 mm diameter is provided in each of the three 500 mm pipes. Instead of three parallel pipes, two pipes can also be used.

When the flow through a large diameter (more than 1000mm diameter) pipeline needs to be controlled then this type of arrangement is important by incorporating a proper flow controlling mechanism apart from the isolation valve.

6.10 Design of Branched Pumping Mains

The branched pumping mains are of two types - direct pumping and combined pumping and gravity system.

6.10.1 Direct Pumping

It may not be possible to feed all ESRs by gravity from MBR/clear water sump at WTP. In that case, it is necessary to locate the sump at the appropriate place and pump water to the needed ESRs (Figure 6.12).

First preference should be given to pump water by separate pumps to separate ESRs by separate pumping main if ESRs are in different directions from the sump. In this case, diameters of the pumping main work out to be less. If this arrangement is not possible, then a branched pumping main as shown in Figure 6.12 is the option.

If the pumping head is not much, it is desirable that combined pumping and gravity mains are used. In a combined system, water will be pumped to an MBR which in turn will supply to ESRs by gravity.

The methodology suggested for the economical design of pumping main to single MBR can be extended for the design of a direct pumped transmission main system feeding to multiple reservoirs (Figure 6.13), or a combined pumping and gravity transmission system. In a direct pumping system, an increase in the pumping head results in a decrease in network cost but increases the cost of the pump, and associated energy cost over its design period. Similarly, in a combined pumping and gravity system, with the increase in the height of MBR, network cost decreases but the pumping cost increases. Therefore, to arrive at the economical diameters in both cases, pipe cost, energy charges, and cost of pumps and other costs, as discussed above, should be considered. Two approaches: (i) using JalTantra software; and (ii) using the hydraulic model are discussed.

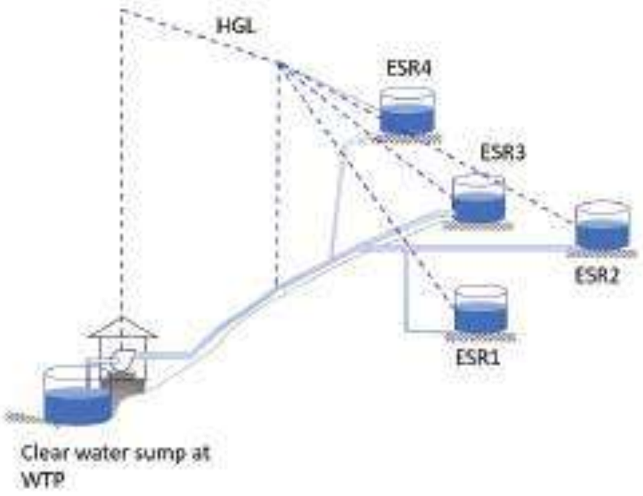


Figure 6.12: Branched Pumping Main

i. **Using JalTantra Software:** In case of a water transmission network (WTN), a single pumping main is now replaced by a network. As several pipes are to be sized and several options are available for each pipe, many combinations can be formed giving different pumping heads and different network costs. Evaluating all these alternatives to select the best alternative is difficult for most practical problems. Therefore, a combination of the Linear Programming based optimisation methodology for the design of branching networks with different source heads and the present worth (PW) method of economic analysis for comparing alternatives is recommended for the optimal design of WTN.

   The entire methodology consists of the following steps.

   a. Consider two stages of 15 years each and calculate the design flow at the end of each stage. Also, find the average flows for both stages.
   b. Select an initial trial value of the source hydraulic gradient level (HGL). This may be obtained by considering an average head loss (say 1.5 to 2 m/km) on the critical path and the minimum required HGL at the critical node including the residual head. (Critical path from source to any demand node is the path having the least available hydraulic slope, and the critical node is the node at the end of the critical path).
   c. Design a WTN for the selected source head using LP for the ultimate stage flow. The JalTantra software or any other LP or ILP-based model can also be used. Obtain the cost of the network. Check pipelines for the velocity and water hammer pressure criteria. Modify sizes or class of pipe, if necessary, and find the revised cost.
   d. Calculate the necessary pumping head for the selected value of the source head and obtain the pump capacity for the ultimate stage and its cost.
   e. Carry out analysis of the network for intermediate stage flows and obtain the necessary pumping head for the intermediate stage. Also, calculate the pump capacity and pump cost. Find PW of the pump cost.
   f. Obtain the number of hours of pump operation for the mean flow during both stages considering the operation of pumps for the required number of hours at the end of the stage.

Calculate average annual energy charges and obtain PW of energy charges at the beginning of their stage.

g. Find the PW of the pipe cost, pump cost, and energy cost. PW of other cost components like operation and maintenance costs can also be obtained in a similar way.

h. Repeat steps (3) to (7) by adding a fixed increment to the source head. If the PW is found to be less than that of the previous alternative, continue further. Else, check by lowering the HGL value of the source head.

Initially, a higher increment can be taken to find an approximate value. Increment can be decreased for obtaining a more correct value.

ii. **Using hydraulic model:** A GIS-based hydraulic model can be effectively used for equalisation of residual pressures at FSL of service tanks. For carrying out the optimisation as well as equalisation, the hydraulic model needs to be prepared which can be prepared using any network freeware software or any commercial software. The advantage of using such software is that the transmission main can be mapped on GIS.

Equalising residual head at FSLs of ESRs is then achieved by a simple method called “moving node method”. By dissipating extra residual head and by bringing residual head to 3 to 4 m for all ESRs/GSRs, the storage tanks receive water just equal to their design requirement.

By equalisation of pressures at FSL of service tanks, a proper timetable of closing inlet valves can be enforced without allowing any stretch of transmission main from getting empty.

Two case studies of direct pumping are presented:

a. Non-spatial rural water supply scheme (RWSS) for multi-villages with optimisation of pipe cost and equalisation pressures at service reservoirs using JalTantra software. A case study of RWSS in Nadia District of West Bengal is presented in **Annexure 6.4**.

b. A GIS-based hydraulic model with optimisation of pipe cost and equalisation pressures at service reservoirs using established software. A case study of the Shirpur water supply scheme in the Dhule district of Maharashtra is presented in **Annexure 6.5**.

6.10.2 Combined Pumping and Gravity System

A combined pumping and gravity system is shown in Figure 6.13. In this system, water from the clear water tank (CWT) is pumped to the MBR, which then supplies water to various service reservoirs by gravity. The objective is to compute the optimum LSL of MBR for which the capitalised value of pipes and energy is the least.

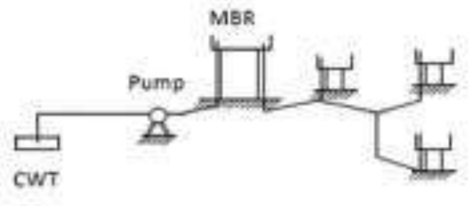


Figure 6.13: Combined Pumping & Gravity System

Optimisation of the cost of pipes and energy is done by using JalTantra software. A case study of one city representing a combined gravity and pumping system is presented in **Annexure 6.6**.

6.11 Interlinking of Transmission Mains from various sources for disaster management

Heavy rainfall causing floods and wash away of intake wells, lack of monsoon causing dried up source, silting of WTP through sand are some of the examples of disastrous condition in which no water is available from an affected source for some period of time. In such a situation, it is desired that water is made available to consumer from other nearby alternative sources. This requires linking of transmission mains from various sources.

6.11.1 Concept of Ring Main in Chennai

The City of Chennai, experiencing frequent draught, had implemented ring main system around its core area. Ring main system receives water from all the sources with an objective to maintain adequate water supply in different parts of the core area of the city in the event of failure of any surface water resource.

A schematic of the water supply from the British era core city of 67 sq. km and its expansion initially to the expanded city and to the present Chennai Metropolitan Area (CMA) of 1189 sq. km, is shown in Fig. 6.14. The Red Hills Lake and its water treatment plant (WTP) gets inflows from Sholavaram Lake and this in turn gets inflows from the distant Poondi Lake (not in the drawing). The Chembarambakkam Lake and its WTP gets water from Veeranam Lake, 235 km down south (not in the drawing). The Poondi and Chembarambakkam Lakes are interconnected by a “level bedded canal” to “balance” the waters in these lakes in floods and droughts. Two seawater desalination plants (DSPs) are on the north and south ends. In the British era, it was only the Red Hills Lake gravitating the water to the city to a ground level reservoir (GLR) and pumped (by steam engine driven pump sets) to ESRs in the then three distribution zones. The later needs were to feed the new distribution zones in extended city and CMA and physically and functionally interconnect. This was done with inputs from the World bank and other local funding institutions. The water from the WTPs, DSPs and other minor sources inject into the ring main along its alignment to keep it as hydraulically floating to facilitate drawls physically and functionally by valve controls to the various zones. The historical GLRs and pumping to ESRs are retained and all new zones are by “flat pumping” directly from GLRs (SUMPs) into their distribution system even from the 1990’s. This ring main system can be adapted in the old walled cities as also newer planning cities to command both inward and outward distribution from the ring main as a decentralised-centralised system.

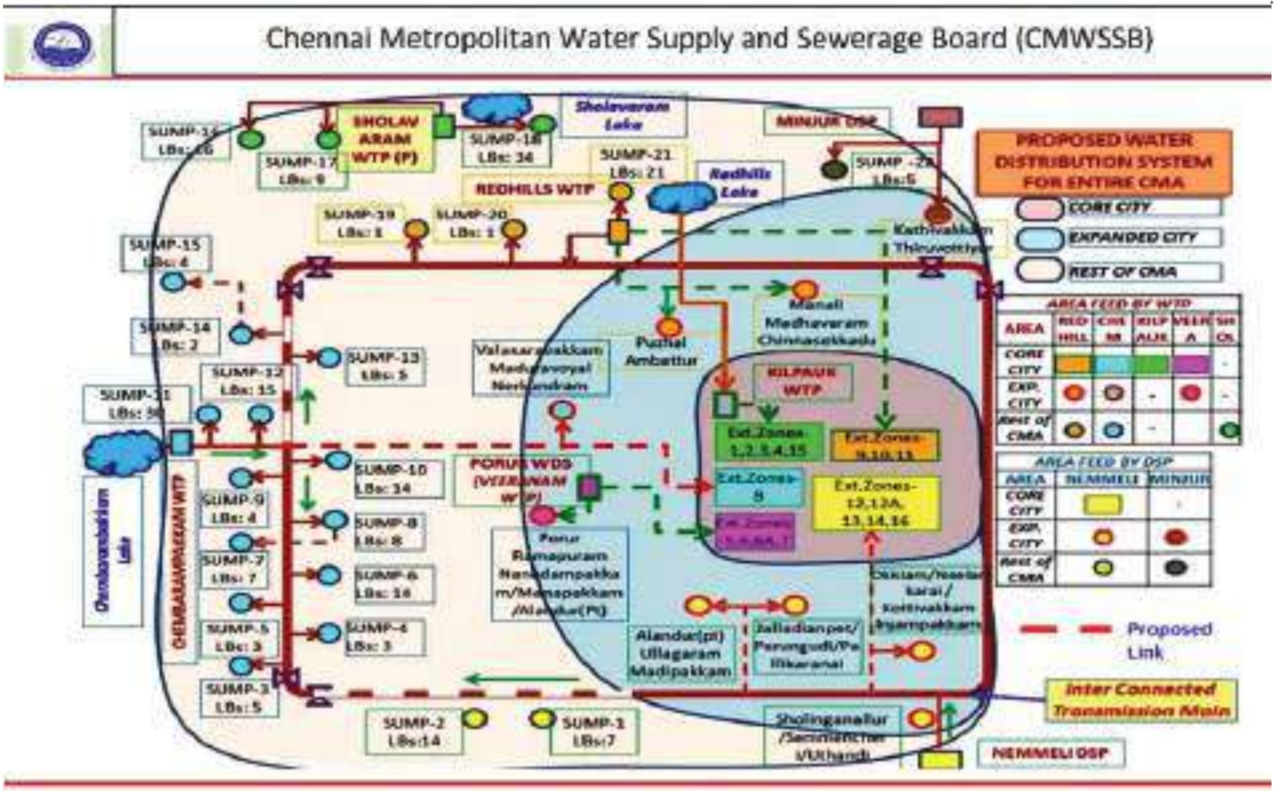


Figure 6.14: Chennai Ring Main Connecting Different Sources

(Source: Authors at Reference no. 7 of Bibliography for Chapter 6)

6.11.2 Interlinking of transmission mains in Mumbai Metropolitan Area

The City of Mumbai experienced a record-breaking 942 millimetres of rain in a period of 24 hours on 26 July 2005. The heavy monsoon rain triggered deadly floods, which had disrupted the water supply scheme. The water supply of suburban towns was totally affected. Following this event, a disaster management plan was prepared and implemented in Mumbai metropolitan area covering 12 cities by interlinking transmission mains from various sources at various locations.

However, it is suggested that if the transmission system around any core area of the city or any other periphery area of the city fails, the concept of ring main may be adopted in a decentralised manner for different areas which are fed by at least two sources so that water will be available even if one source fails. Full supply from an alternate source cannot be guaranteed, however, the availability of 20% to 40% of supply from an alternate source can be planned.

6.12 Surge Protection for Pumped Transmission Mains

Pumped transmission mains should be checked for water hammer analysis by any established software. A sample result of one such analysis is shown in Figure 6.15.

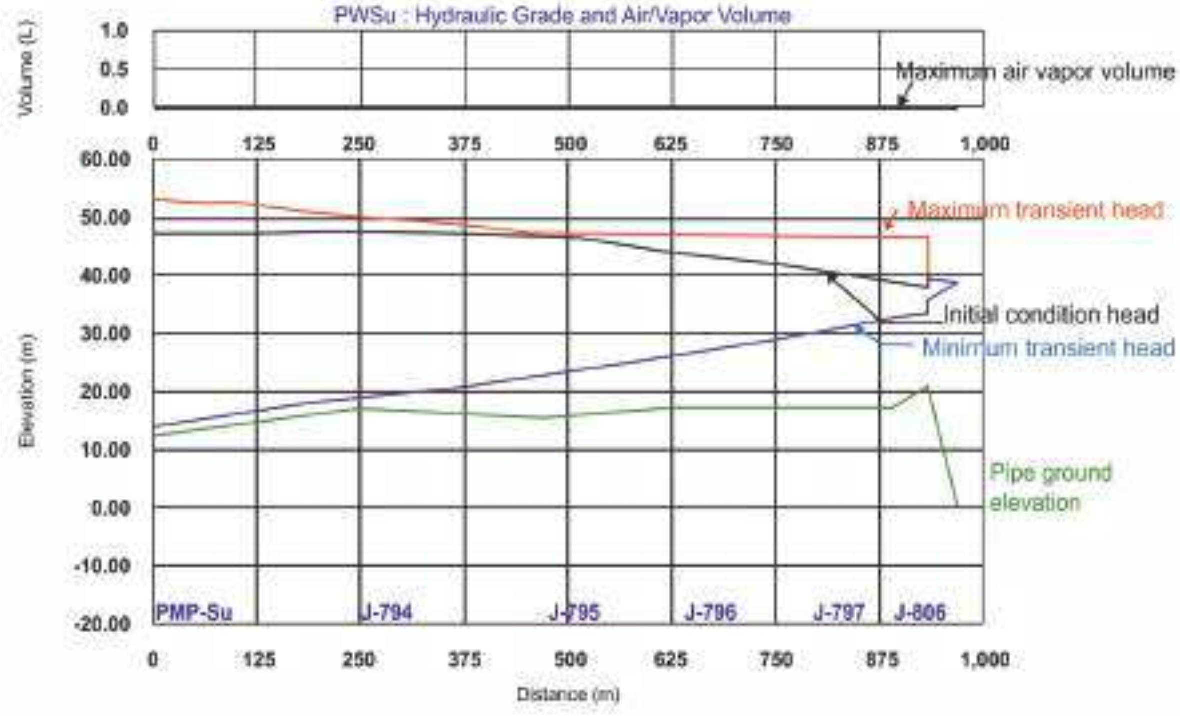


Figure 6.15: Sample Result of Water Hammer Analysis of a Pumping Main

It is to be observed that the minimum transient headline (shown in blue colour) must be above the ground elevation line (shown in green colour) of the pumping main which indicates that the pumping main is safe from cavitation. The pipe class should be such that it sustains maximum transient head shown in red colour. If the minimum transient headline (blue colour) happens to be below that of the ground elevation line (green colour) then the pipeline is unsafe. In such a situation, water hammer protective equipment should be designed.

6.13 Minimisation of Energy Cost

Normally, the side water depth of MBR is 5 m, the inlet is at FSL, and the outlet is at LSL. However, it is recommended to keep invert levels of inlet and outlet at the same level, and the bottom of MBR with a non-return valve. LSL of MBR is lowered down to the extent possible. The bottom of MBR is placed further 1 m below the designed LSL of MBR as shown in Figure 6.16.

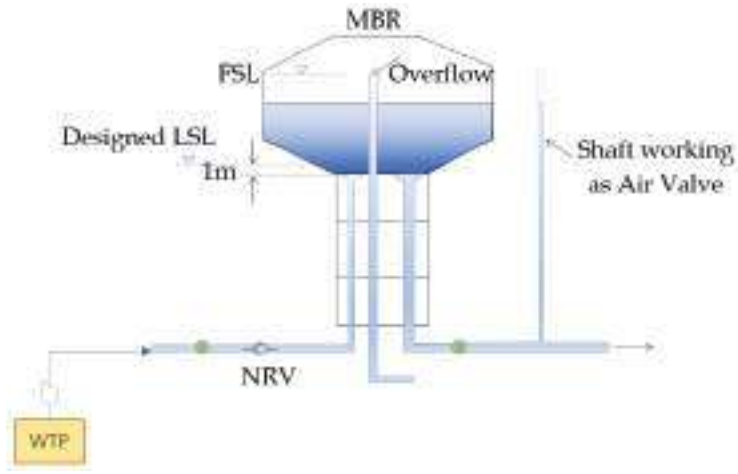


Figure 6.16: Inlet, Outlet Arrangement of MBR

This arrangement saves energy. We can save energy cost of pumping head perpetually, i.e., every month.

In case rising main to MBR leaks, then wastage of water due to emptying of MBR can be saved by shutting the pumps and closing the valve at the inlet.

In the case of MBR, if located on a hillock, i.e., on ground level, then the outlet of MBR should be with a bell mouth embedded below the bottom so that full capacity is available for use and MBR can be cleaned during maintenance. It is necessary to have an all season road to MBR/BPT. Overflow pipe from FSL should discharge water at a place away from MBR and then that discharge should find its way to the natural stream.

6.14 Break Pressure Tank (BPT)

6.14.1 Merits of Introducing BPT

If a long pumping main encounters a hillock at a high altitude such that discharge on the downstream side of hillock/high-level ground can flow by gravity, then in such case advantage of topography can be taken by introducing a tank as BPT at such hillock. Even if high-level terrain is encountered such that HGL at the high-level ground is within 20-25 m above ground level, BPT can be introduced. The provision of BPT renders advantages as follows. Refer to Figure 6.17.

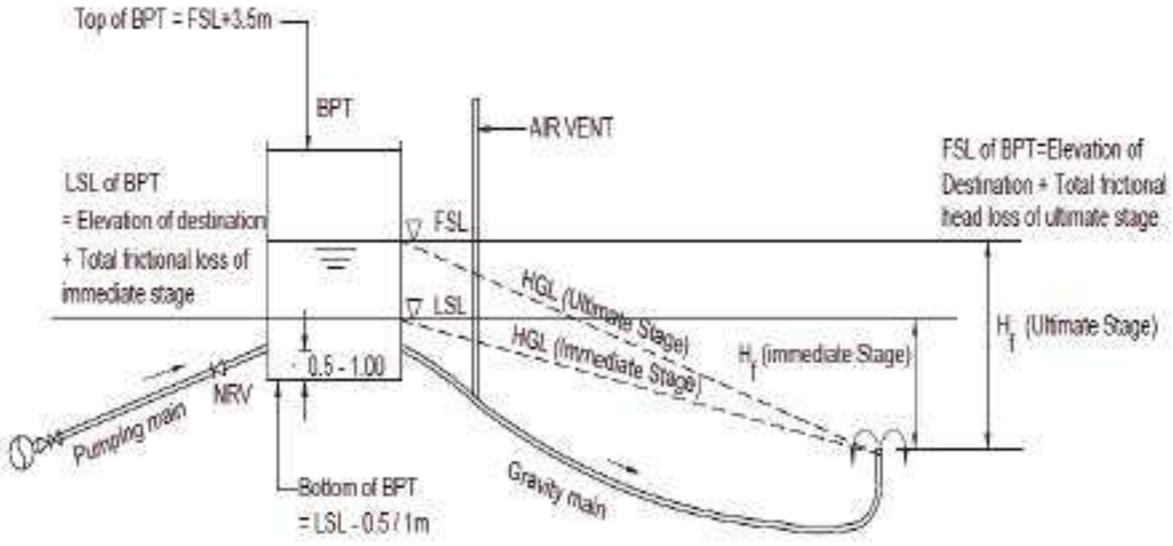


Figure 6.17: General Arrangement of Break Pressure Tank

i) No BPT Case

- In the absence of a BPT, the entire transmission from the pumping station to the destination would have functioned as a pumping main and would have to be designed for design pressure equal to the sum of operating pressure and water hammer pressure.
- The cost of such a high pressure pipeline shall be very high.
- The pipeline section at hillock becomes a critical stretch for sub-atmospheric pressures and consequent water column separation.

To overcome this critical aspect, well-designed and dependable water hammer protection device becomes essential to prevent the collapse of the pipeline due to sub-atmospheric pressure. The cost of such a water hammer protection device is usually high.

ii) On the introduction of BPT at hillock/high ground

- Due to BPT, the downstream pipeline functions as a gravity main. Thus, the downstream pipeline shall be totally free from water hammer pressures. A lower class of pipeline or lower thickness can be selected resulting in large savings in capital cost.
- The length of the pumping main is reduced from the pumping station to BPT. The cost of a water hammer protection device for reduced length of pumping main shall also be less particularly as a critical section on hillock vulnerable to sub-atmospheric pressure and water column separation is no more applicable due to locating BPT at such section.

6.14.2 Improvisation by Manipulating BPT Location

It is not necessary that BPT location at intermittent hillock or high ground is a must. If suitable hillock or high-level ground is available at a short distance from the pumping station, such that HGL at such high ground is within 20-25 m above ground level, BPT can be introduced at such place. This arrangement converts the maximum length from the pumping main to the gravity main.

Figure 6.18 shows the theoretical location of a BPT on an enroute hillock at 15.5 km out of a total 56.5 km transmission main due to which 15.5 km becomes the pumping main and 40 km as the gravity main.

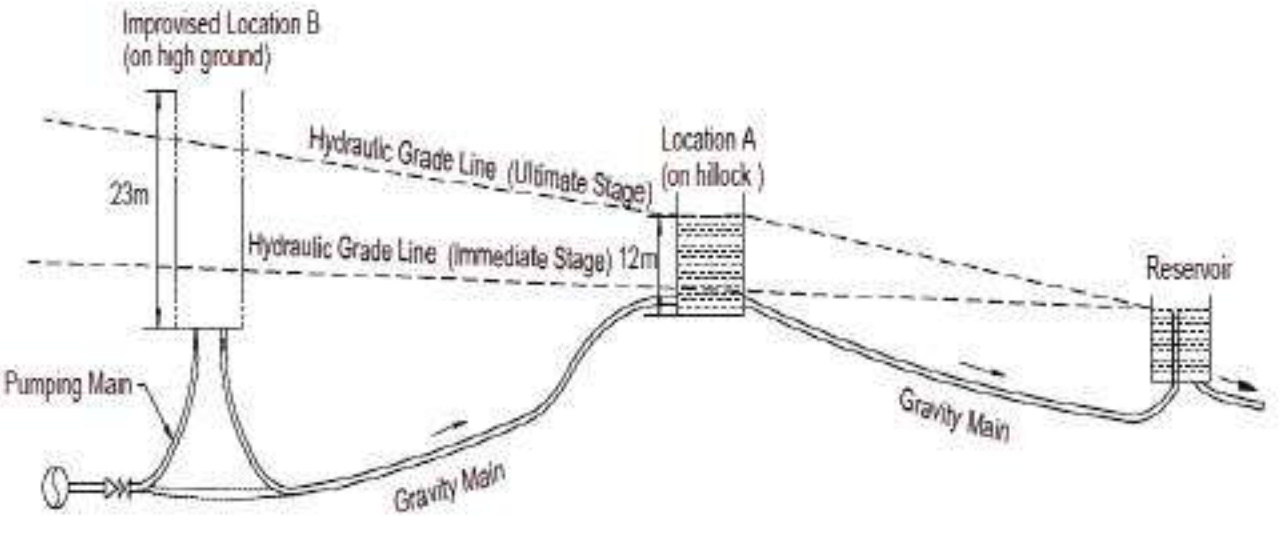


Figure 6.18: Improvised Location of BPT for Increasing length of Gravity Main

On improvisation by application of the principle, in the scheme for the city, a revised location of BPT is kept at the nearby high ground at a chainage of 1.5 km. A BPT of 8.6 m diameter × 23 m height is constructed (Figure 6.19) due to which the length of pumping main is now reduced to 1.5 km and 55 km length functions as gravity main.

In another scheme with a pump head of 56 m and 600 MLD flow, the entire 40 km long transmission main is laid in plain terrain. The advantage of the availability of hillock is availed for locating BPT near the pumping station. Due to BPT, 39.8 km pipeline functions as gravity main and length of pumping main reduced to a mere 200 m. Thus, a significant savings in capital cost of pipeline, as well as water hammer control device, could be achieved.

Figure 6.19: 23 m high BPT nearing Completion

6.14.3 Usual Mistakes in BPT Design

i) Common Observation on Capacity: It is observed that in the absence of guidelines, often the size of BPT is arrived at from consideration of the volume required to store water at a steady design discharge for an arbitrary period. The arbitrary period is decided based on the experience of the designer, which could be a wild guess like 5 minutes, 10 minutes, or 15 minutes. Because of the fear that the size may become inadequate, BPTs of much larger sizes than required are provided in various schemes.

The cross-sectional area of a BPT can be calculated, and if guidelines are followed, a BPT of much smaller capacity ranging from 2 minutes to 10 minutes can be adequate. Many BPTs designed, as per the guidelines, are functioning. A detailed discussion is as follows.

ii) LSL and FSL: Another usual mistake in designing BPT on a similar basis applicable for service reservoir, keeping LSL with friction losses for the ultimate stage. The result is that BPT admits and passes full flow at LSL only and the BPT runs practically dry. Hence, LSL is to be designed considering Hazen-Williams' 'C' for new pipes and FSL is to be designed for friction losses for Hazen-Williams' 'C' values for the old pipe.

iii) Incorrectly terminating inlet pipe at FSL: Similar to the service reservoir, the inlet pipe is terminated at FSL. In the initial stage, WL in BPT is at LSL. Due to the termination of the inlet pipe at FSL, the pump discharges at FSL whereas WL in the tank is at LSL, resulting in an unnecessary increase in the pump head equal to the design water depth of the tank. Hence, both inlet and outlet pipes shall be terminated at the LSL of BPT.

iv) Misunderstanding about Q_{in} and Q_{out} and balancing storage: In BPT, Q_{in} and Q_{out} are always the same irrespective of demand in the distribution system. In the case of ESR, Q_{in} is always constant, but Q_{out} varies from 20% to 250%-300%, depending on lean hour and peak hour demand. Hence, balancing storage as per the mass diagram is provided in the service reservoir. However, balancing storage in BPT is not applicable.

v) Misconception about the increase in pump head due to BPT: Generally, the misconception is observed that due to the introduction of a BPT, pump head increases. There is practically no change in HGL as well as pump head due to the introduction of BPT, as the inlet is kept at the level of the outlet. Only an exit loss at the inlet and entrance loss at the outlet is added, the magnitude of which are very low - about 0.1-0.2 m, which is insignificant.

6.14.4 Hydraulic Design of BPT

Design objectives of BPT can be stated as follows:

i) BPT should never overflow during the starting of pumps and normal steady state operation over the entire service period of BPT from the initial stage when the 'C' value is better, immediate stage, and ultimate stage when the 'C' value is the lowest due to deterioration.

ii) During starting of the pump, when standstill water in the downstream pipeline starts flowing, velocity is accelerated from V=0, causing WL to rise until steady state velocity, $V_{o,}$ is attained. During this acceleration period, WL attained may be higher than steady state WL. Even under this period, overflow should not occur.

iii) Under no circumstances should the head-on pump be wasted. This objective can be achieved by terminating the inlet and outlet at same level as discussed in subsequent subsections.

iv) BPT should never be dry or fully empty. Generally, the tank is in RCC or steel construction. Concrete deteriorates if dry and steel tanks get corroded if subjected to dry and wet situations.

Design aspects

(i) Variations in design basis

The design of BPT depends on the profile of the downstream pipeline, the water content in the pipeline under standstill conditions achieved after stoppage of pumps (usually called no-flow condition), and flow characteristics during starting of pumps in multi-pump installation.

(ii) Categories of gravity main on the downstream side of BPT

The pipeline on the downstream side of BPT, i.e., gravity main, can be classified into three categories depending on the characteristics of the pipeline which include the longitudinal profile of the pipeline, average slope of the pipeline, and slope of hydraulic grade line (HGL).

a. Category-I: Refer Figure 6.20:

When the average slope of gravity main is greater than the slope of HGL, some length of pipeline from BPT will run partially full. BPT will remain empty all the time. Providing large size BPT, in this case, is not required and BPT with the nominal size is enough. In order to ensure that BPT is not dry, the outlet should be kept at least 0.5 m above the bottom of the tank.

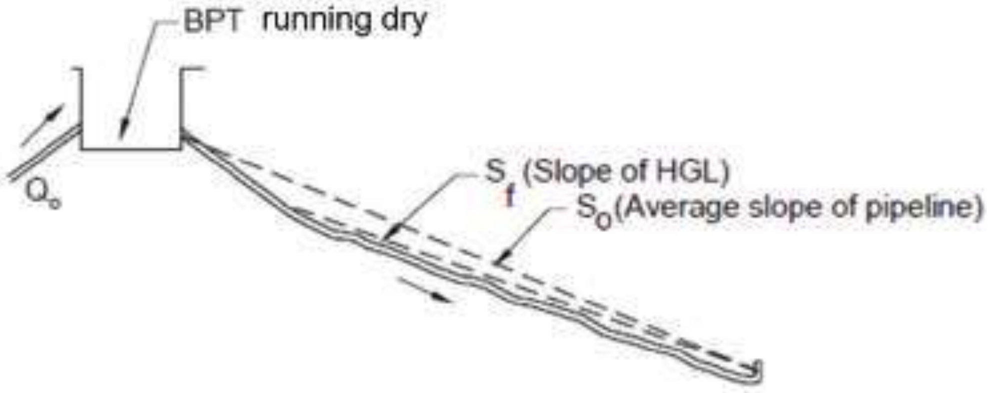


Figure 6.20: Category-I: S_0 (Average slope of pipeline > S_f (Slope of HGL)

b. Category-II: Refer Figure 6.21

In another case, the average slope of gravity main is less than the slope of HGL, and the longitudinal profile of the pipeline is such that during no-flow, the pipeline remains empty as the water is drained out due to a continuous downward slope after stopping inflow into BPT. In this case, when the inflow to BPT starts, water enters the pipeline and process of filling up of pipeline begins and the water level in the pipeline starts rising. Simultaneously, the velocity of water in the pipeline increases gradually. Thus, the water level will reach a steady state position gradually and will remain stationary at that position. In this case, a large size BPT is not required; nominal size is enough.

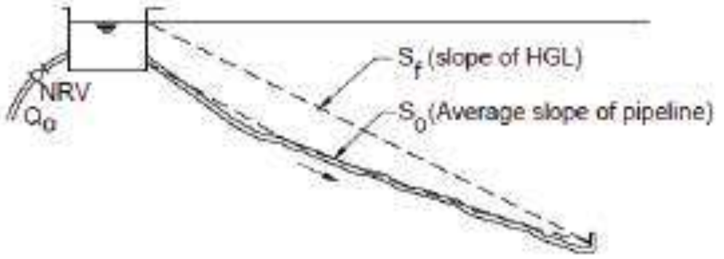


Figure 6.21: Category-II: S_o (Average slope of pipeline < S_f f (slope of HGL)

c. Category-III: Refer Figure 6.22

In the third case, the average slope of the pipeline is less than the slope of the hydraulic grade line (HGL), but the longitudinal profile is in the form of an inverted siphon. The pipeline remains practically filled with water after the stoppage of pumps.

This case is vital for detailed design and therefore, elaborated covering all pertinent design aspects.

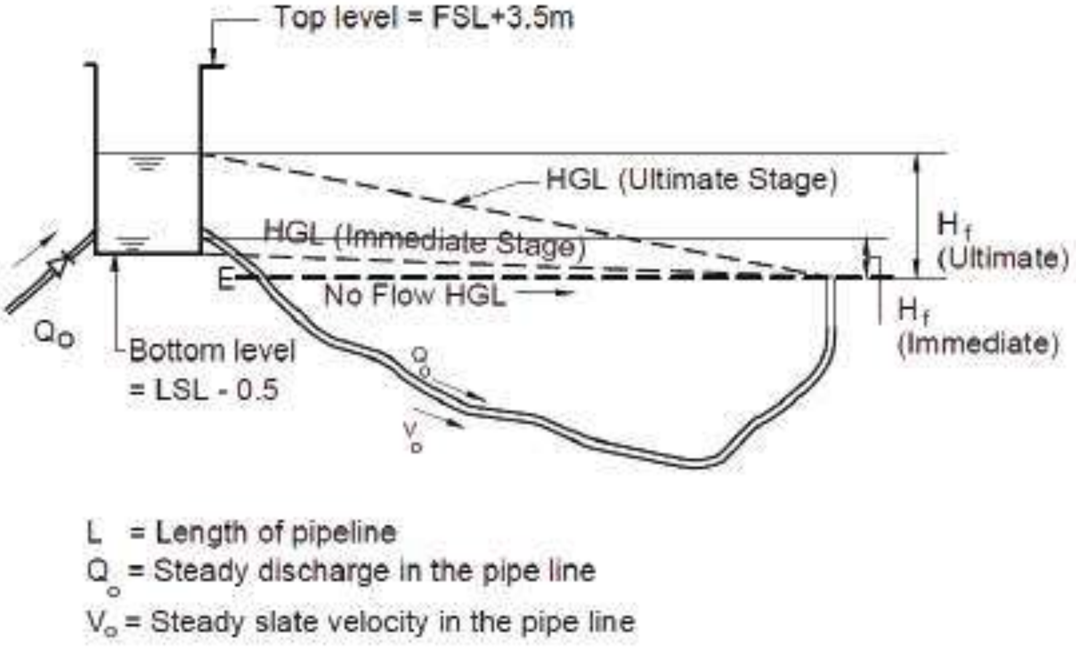


Figure 6.22: Category-III: Pipeline in form of inverted siphon

(Pipeline practically full under no-flow conditions)

(iii) Terminating inlet and outlet pipes in BPT

The outlet of the pumping main (which is inlet of BPT) and inlet of the gravity main (which is outlet of BPT) shall be kept at the same level and should be marginally above the bottom of BPT as shown in Figure 6.21.

This will save additional head on the pumps which otherwise would have come if the outlet of the pumping main is kept above FSL of BPT. By this arrangement, the advantage is that the water level in the BPT can rise to such a level that the driving head is just sufficient to negotiate the frictional losses occurring in the gravity main for the immediate stage and also the ultimate stage. This will save energy costs for both the immediate stage and ultimate stage. The water level will increase just to the required level and the energy cost can be saved.

Usually, the top of the inlet is kept at FSL in the reservoir on the reasoning that if a burst or major leakage occurs in the pumping main, water in the reservoir should not drain causing water logging at the leakage location. This reasoning can be acceptable for service reservoirs or MBRs where capacities are four to eight hours. In BPT, however, capacity is much less, i.e., few minutes.

However, even to prevent such draining of BPT, a non-return valve (NRV) can be provided on the incoming pipeline to allow inflow to tank; but prevent reverse flow as shown in Figure 6.20. Head loss in NRV is 0.15-0.3 m which is insignificant compared to the average saving in pump head by about 3-5 m.

(iv) Deciding the Lowest Supply Level (LSL) of BPT

Initially, the driving head required to pass the intermediate stage (base year+15) flow through the gravity main is required to be computed which is equal to the elevation of the destination, in this case, the elevation of the lip of the aerator plus the total frictional head (including minor losses) of intermediate flow as shown in Figure 6.21. Care should be taken that while working out the frictional head loss, the C-value of the new pipe (highest C-value) should be taken. Thus, the LSL of BPT is decided. The bottom of BPT should be a minimum of 0.5 m below LSL to ensure that the tank never remains dry.

(v) Deciding FSL and Height of BPT

Initially, the frictional head loss (including minor losses) for the ultimate flow that the gravity main can pass should be computed. Care should be taken that while working out the frictional head loss, the C-value of the old pipe (lowest C-value) should be taken. FSL of BPT is then elevation of destination plus the frictional head due to ultimate flow (including minor losses). Considering the safety of 2 m against overflowing, the height of BPT is then computed as,

Top of BPT = FSL + 2.5 m (including free board of 0.5) (6.8)

(vi) Area of cross-section of BPT

V.N.I.T., Nagpur has developed guidelines for sizing BPT, based on the equation of continuity and equation of motion, the equation for the cross-sectional area of BPT is developed which is given by:

$$A_T = \frac{4AL}{F^2 V_o^2 g} \qquad (6.9)$$

Where: A_T = Cross-section area of BPT; A = Cross-section area of downstream gravity pipe; D = diameter of gravity pipe; F = fL/(2gD) = friction loss constant; g = gravitational acceleration; L = length of pipeline; V_0 = steady state velocity in the pipeline.

$$h_f = FV_o^2$$

Or,

$$F = \frac{h_f}{V_o^2} \qquad (6.10)$$

Here h_f can be computed using the Hazen-Williams Formula.

Optimisation of BPT can be done by reducing A_T (cross-section area of BPT) by 20%-30% in which case, small WL rise above steady state WL may occur. However, this small rise can be accommodated in a safety margin kept above FSL.

In essence, the following should be adopted for inlet and outlet pipes for all above three cases:

a) Inlet and outlet should be kept at the same elevation.

b) LSL of BPT should be computed for present stage demand with C-value of new pipes and FSL is computed with ultimate stage demand and C-value of the old pipe.

c) Every design including hydraulic modelling always has a factor of safety. In the design of the pipeline, the factor of safety is in terms of a slightly higher designed LSL of MBR. Design LSL is of course not to be lowered down but unnecessary pumping costs can be saved. This can be done by providing the bottom of the slab at an elevation lower by 1 to 2 m below the design LSL. In the steady state of operation, i.e., inflow equal to outflow, the water level will not climb up to the designed LSL but will remain at a level lower than that and the pumps will operate for this decreased head. This yields in saving on electricity bills due to a decrease in the head of the pump by more than 5 m. The decrease in the head due to this arrangement compared to the inlet at FSL is 5 to 7 m. This is an extra saving over and above saving. If the head of the pump on the inlet pipe is 50 m, then the saving is about 10% to 14%. For lengths of transmission mains up to 10 km, provide the bottom of MBR at 1 m below the design LSL and for more lengths bottom of MBR should be 2 m below the designed LSL.

A typical design of BPT is illustrated in **Annexure 6.7** using the data of the water supply scheme of one city.

6.15 Thrust Block

It is necessary to provide thrust block (Figure 6.23) in the shape of concrete blocks to resist the forces that cause the pipe to pull apart at bends or other points of unbalanced pressure or when they are laid on steep gradients and resistance of their joints against longitudinal stresses is either exceeded or inadequate. Adequate anchor bars must be provided as per the site conditions embedded in concrete blocks to give additional strength and stability.

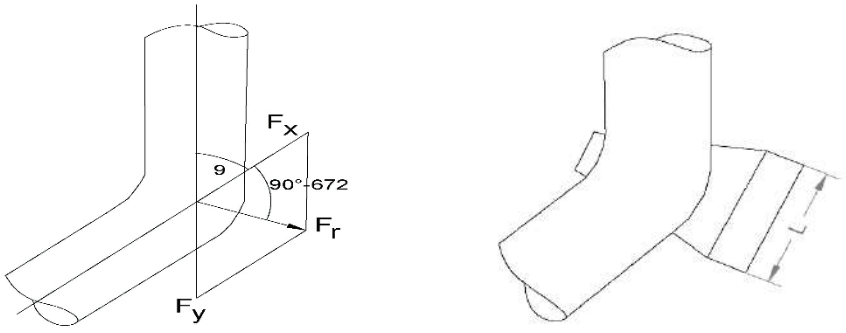


Figure 6.23: Thrust at a Bend & Thrust Block

Thrust blocks made of concrete, generally in rectangular shape, resist the unbalanced horizontal thrust to pull out the bend or pipe by counteracting the following forces:

(i) Weight of the block + weight of water in the enclosed pipe in the block
(ii) Friction resistance by soil
(iii) Lateral pressure acting on the block by soil mass
(iv) Lateral resistance of soil mass on the outer face of the projected pipe

Horizontal thrust caused by unbalanced static pressure by water at the bend,

$$F_p = 2PA\sin(\phi/2) \tag{6.11}$$

Where P = Internal water pressure in the pipeline

A = Area of cross-section of pipe

ϕ = Degree of bend angle

Counteracting forces to resist the horizontal thrust

It is as below:

(i) Weight of concrete block = Length × Breadth × Height × weight of concrete/unit volume
(ii) Weight of water in the pipe enclosed in Cement Concrete block = cross-section area of pipe × Length of pipe × wt. of water/unit volume
(iii) Weight of earth cushion over the concrete block= Width of block × height of the earth cushion × pipe diameter × weight of earth/unit volume

The lateral resistance offered by soil friction against the thrust block = (A + B + C) × Frictional resistance of soil

Lateral resistance of soil against the thrust block,

$$F_p = \gamma_s \frac{H^2}{2} L \left[\frac{1+\sin\theta}{1-\sin\theta}\right] + 2CHL\sqrt{\frac{1+\sin\theta}{1-\sin\theta}} \quad (6.12)$$

The maximum resisting pressure a soil mass will offer is termed the passive resistance and is given by:

$$f_p = \gamma_s h \left[\frac{1+\sin\theta}{1-\sin\theta}\right] + 2C\sqrt{\frac{1+\sin\theta}{1-\sin\theta}} \quad (6.13)$$

This maximum possible resistance will only be developed if the thrust block is able to move into the soil mass slightly. The corresponding maximum soil pressure is termed passive pressure. The minimum pressure which may occur on the thrust block is the active pressure, which may develop if the thrust block were free to yield away from the soil mass.

$$f_\alpha = \gamma_s h \left[\frac{1-\sin\theta}{1+\sin\theta}\right] - 2C\sqrt{\frac{1-\sin\theta}{1+\sin\theta}} \quad (6.14)$$

F_p, f_p = Lateral resistance of soil against the thrust block; γ_s= soil density; h = depth in m, θ = angle of friction in degrees, C = cohesion of soil (C = 0 for gravel and sand, 0.007 for silt, 0.035 for dense clay, and 0.15 for soft saturated clay), H = height of thrust block and L = the length of thrust block

Total counteracting forces by concrete block at bend should be ≥1.5. For the safe design of the thrust block, the factor of safety is 1.5. The minimum reinforcement in all thrust blocks should be provided 5 kg/m2. The spacing of these bars should not exceed 500 mm c/c.

In the case of end caps, either a thrust block at the end cap is required or the end cap should be dish-shaped like the ends of the air vessel.

A typical design of thrust block is given in **Annexure 6.8**.

Anchorages for Sloping Pipelines

Thrust block on slopping ground (Figure 6.24) is described by a step-by-step design guide (Thorley, 1994) for thrust blocks. It mentions restraining the forces generated by changes in direction of fluid flow in joint buried pressure pipeline networks.

Where buried pipes are laid in a straight line on slopes, a component of the dead weight of the full pipeline acts axially, increasing with the angle of the slope. This axial force pushes the pipes to slide down the slope. The design should prevent such movement from occurring.

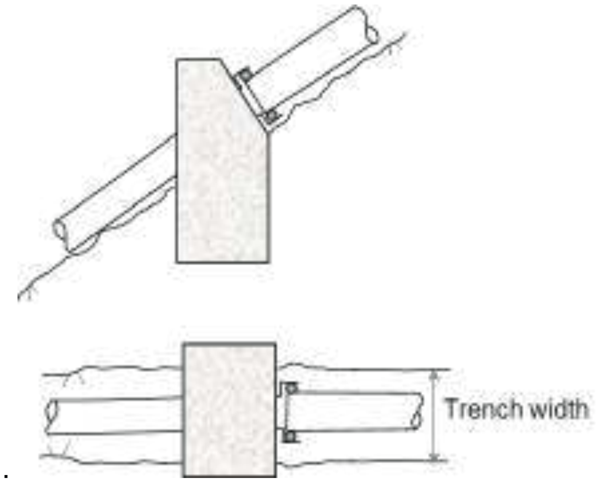


Figure 6.24: Typical Thrust Block on Sloping Ground

Pipes laid on shallow slopes do not slide due to the frictional resistance of the soil. However, if the pipeline is loosely wrapped with a polyethylene sleeve, the resistance becomes less and there is a chance to slide. Also, when slopes are such that it generates a sliding force more than frictional resistance, the pipeline must be supported with concrete anchors with integral keys or even by raking piles for slopes more than 1 in 4.

Concrete walls surrounding the pipe should extend at least half the pipe diameter above the crown and below the underside of the pipe and beyond the trench walls into the undisturbed ground on either side and be of suitable thickness to develop the required bond and to accept the shear and bending moments generated (Figure 6.24).

Pipes should be laid with their sockets facing uphill, and support structures located so that the external shoulder of the socket of each pipe bears against the pipe support. In this way puddles, flanges, or other securing devices are not required. Each pipe should be anchored. The use of anchored or self-restrained joints as an alternative should be considered irrespective of the pipe materials.

Proper attention should be given to preventing the erosion of the bedding material beneath the pipe. On long slopes, and depending on the gradient, more than one thrust block will be required. Table 6.8, taken from recommendations by Stanton Pipes for cast iron pipelines, gives spacing for the thrust blocks.

Table 6.8: Spacing for Thrust Blocks on Long Slopes

Gradient	Spacing for Thrust Blocks
1 in 2	5.5 m
1 in 3	11.0 m
1 in 4	11.0 m
1 in 5	16.5 m
1 in 6	22.0 m

Source: *(“Guide for thrust blocks for buried pipelines,” A.R. D. Thorley and J.H. Atkinson, published by CIRIA in conjunction with T. Telford, London, 1994)*

6.16 Surge Phenomenon and Selection of Surge Protection Devices

6.16.1 Occurrence of Surge and Causes

If discharge in a pipeline suddenly or rapidly changes, causing sudden or rapid changes in flow velocity, consequently a pressure wave occurs which propagates in the pipeline at acoustic speed both in forward and reverse directions. It lasts till the wave dies due to friction in the pipeline. Due to the propagation of pressure waves both the phenomenon, i.e., pressure drop due to down surge and pressure rise due to upsurge, occur in succession in the pipeline. The pressure drop and pressure rise are termed as surge pressures or water hammer pressures.

The change in discharge can be caused by the following operations or events:

i) sudden/abrupt opening or closing of the valve;
ii) power failure to pump motor sets either due to electric supply interruption or tripping of breaker/fuse failure on incoming switchgear;
iii) sudden stoppage of one pump in multi-pump installation due to any reason, may be tripping of power supply or motor stalling;
iv) starting or stopping of first and subsequent pumps in multi-pump installation;
v) sudden falling of gate of sluice valve installed in-line.
vi) due to the slamming of check valve.

Out of the above causes, causes (i) and (iv) can be controlled and causes (v) and (vi) can be prevented by following suitable procedures as discussed in the sections below.

However, power failure and single pump sudden stoppage are beyond control. These two cases are very critical as discussed below.

6.16.2 Effects of Surge Pressure

The surge pressure wave travels and subjects piping system and other facilities to cycles of transient high and low pressure occurrences. These pressures and phenomenon can have several adverse effects on the piping system. If the transient pressure is extremely high, the pressure rating of the pipe may be exceeded causing failure through the pipe or joint rupture. Such a flow variation causing pressure can also lead to significant pressure reduction during wave travel in forward and reverse directions. If sub-atmospheric pressure condition results, the risk of pipeline collapse increases. Even if the pipeline does not collapse, column separation could occur if the pressure in the pipeline is reduced to the vapour pressure of the liquid. This causes the formation of vapour pockets which collapse when two separated water columns rejoin at high velocities. The collapse of the vapour pocket/cavity can in turn cause severe high pressure and rupture in the pipeline.

6.16.3 Preventing Surges in Starting and Stopping Operation of Pumps and Valves

The basic criterion in surge control is that rate of discharge change shall be such that the operation time of the valve is greater than Tc, i.e., 2 L/c. Here, L = Length of pipeline; c = Pressure wave propagation speed; Tc = Critical time for wave travel in forward and return directions.

Operating procedure as under shall be followed to prevent surges.

i) Delivery valve, whether sluice valve or butterfly valve, should be opened or closed slowly with a uniform speed of opening/closing so as to exceed the time of closing or opening above 2L/c.

ii) The operating speed of the valve actuator, electric or pneumatic, shall be slow to prevent rapid opening or closure and exceed time above 2L/c.

iii) Second and subsequent pump should be started or stopped in sequential order after allowing adequate time for previous pump operation to steady state condition and checking that the pressure gauge reading is steady (usually 10-second time interval per km length of pumping main is adequate).

iv) Overcurrent relay setting and/or fuse rating of incoming breaker and/or switch fuse unit shall be checked periodically.

Starting and stopping of pumps and opening and closing of the valve can be controlled and thus their ill effects can be avoided. However, power failure is beyond control and hence the suitability of pipeline and appurtenances need to be appropriately designed or protection devices provided.

v) Sudden falling of gate can be prevented by periodical checking of line valve/sectional valve.

6.16.4 Magnitude of Surge Pressure

The magnitude of surge pressure is additive/deductive to and from the normal pressure in the pipe and depends on the elastic properties of the liquid and the pipe and the magnitude and rapidity of change in velocity.

Maximum surge pressure (which occurs at the critical time of closure Tc or any time less than Tc) is given by the expression,

$$h_{srg} = cV_o/g \tag{6.15}$$

Where, h_{srg} = maximum pressure rise or fall (upsurge and down surge) in m; c = speed of pressure wave propagation in m/s (also called celerity); g = acceleration due to gravity in m/s^2; V_o = normal velocity in the pipeline in m/s

Speed of pressure wave propagation is given by,

$$c = \frac{1425}{\sqrt{1+\frac{kd}{Et}}} \tag{6.16}$$

Where, k = bulk modulus of water (2.07×10^8 kg/m^2), d = diameter of pipe in m, t = wall thickness of pipe in m, and E = modulus of elasticity of pipe material in kg/m^2 (Refer Table 6.9 below).

Table 6.9: Values of E for Different Materials

Material	E ($Kg/m^{2)}$
Polyethylene - soft	1.2×10^7
Polyethylene - hard	9×10^7
PVC	3×10^8
Cast iron	7.5×10^9
Ductile iron	1.7×10^{10}
Wrought iron	1.8×10^{10}
Steel	2.1×10^{10}
Asbestos cement	3×10^9
Concrete	2.8×10^9
Reinforced cement concrete	3.1×10^9
PSC	3.5×10^9

If the actual time of closure T is greater than the critical time Tc, the actual surge pressure is reduced approximately in proportion to Tc/T.

Surge pressure wave speed may be as high as 1,370 m/s for a rigid pipe or as moderate as 850-1,100 m/s for a steel pipe, and for polyethylene and PVC pipes, may be as low as 200-400 m/s.

6.16.5 Resultant Pressure on Occurrence of Surge Pressures

As stated in 6.16.1, the surge pressure can be down surge and/or upsurge. The surge pressure is subtractive from operating pressure as well as additive and occurs in succession.

Resultant pressure, H_{max} / H_{min} in the pipe system is thus:

During down surge; $H_{min} = H_o - h_{srg}$ (subject to vapor pressure limit)

During upsurge; $H_{max} = H_o + h_{srg}$

Where:

H_{min} = Resultant pressure during down surge;

H_o = Normal/Operating pressure;
H_{max} = Resultant pressure during upsurge

H_{min} however cannot fall below water vapor pressure level as the water vaporises. Refer to 6.16.6 (a) below for further discussion.

6.16.6 Surge Phenomenon due to Power Failure on Pumps

This is a most critical and key surge phenomenon and surge analysis, and selection of surge protection devices aim at protection from effects of down surge and upsurge for this vital event. When the power supply fails, the motor speed reduces rapidly. The rate of speed reduction depends on steady state torque and inertia of the pump motor set. A small pump motor set decelerates very rapidly, whereas the rate of deceleration is slower in the case of a large pump motor set. Consequent to a reduction in motor speed, Q and H also reduce generally following affinity laws. Due to head drop, a down surge pressure wave travels along the pumping main towards the discharging end at wave speed, c. At discharging end, forward flow velocity V_o becomes zero, and subsequently reverse flow occurs at velocity $- V_o$. Simultaneously, the wave gets reflected due to the prevailing atmosphere at discharging end (reservoir or aeration fountain or inlet channel), changes from down surge to normal H (static), and travels towards the pump end at speed c. Consequent to reverse flow, NRV at the pump closes, thus disallowing reverse flow which causes pressure rise, i.e., upsurge.

It is thus seen that at $T = 0$, down surge occurs, and at $T = 2L/c$, upsurge occurs causing surge pressure rise at the pump end. This wave now travels towards discharging end where it gets reflected again at $T = 3L/c$ and pressure reduces to normal H. The surge wave further travels towards the pump end and reaches the pump end at $T = 4L/c$; thus, completing a full cycle. The pressure wave keeps on traveling in a cyclic manner until it dies due to friction in the pipe surface and water.

The magnitudes of the first down surge and first upsurge are maximum and are, therefore, focus points for analysis without protection and selection of water hammer protection device or multiple devices and analysis with the device(s). Figure 6.25 shows maximum and minimum surge gradients without and with protection devices. It is seen from the figure that sub-atmospheric pressures occur at two locations under no protection case and maximum pressure is very high. With a surge protection device, the sub-atmospheric pressures at both locations are prevented and maximum pressure is also reduced.

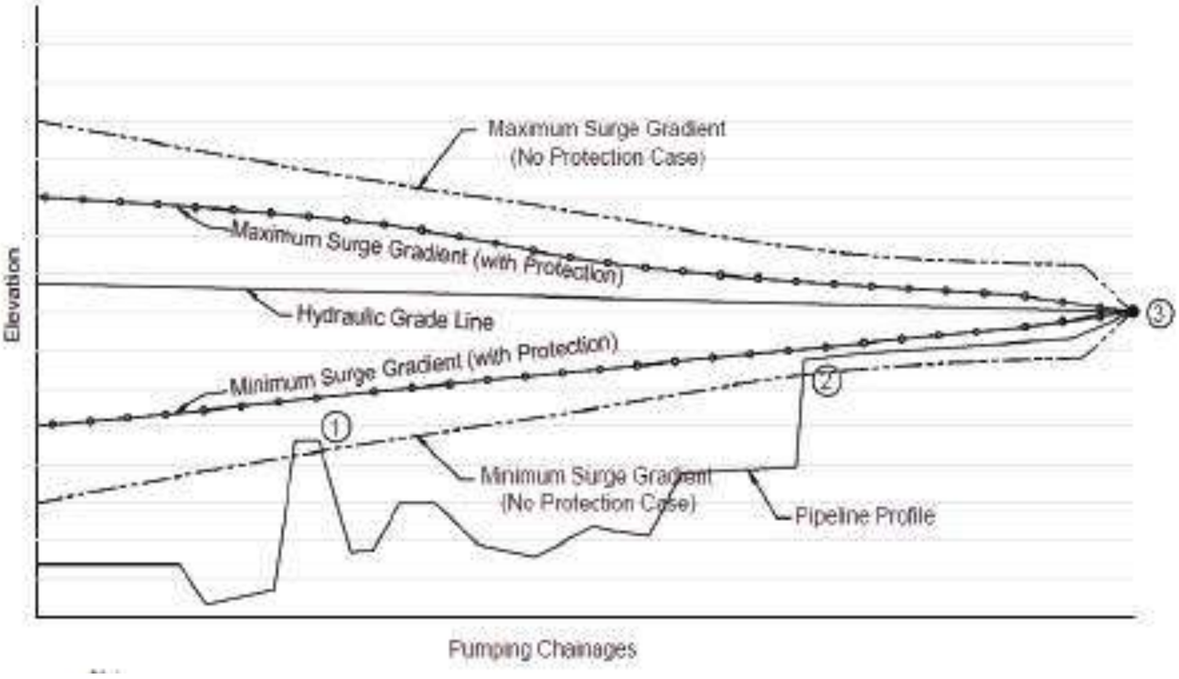


Figure 6.25: Pipeline Profile and Maximum and Minimum Surge Gradients without and with Protection

Note:

1) Peak 1 and pipeline section 2-3 are subjected to Sub-Atmospheric pressure without protection.
2) Due to protection, min surge gradient is above peak and hump section preventing sub-atmospheric pressures.

Both down surge and upsurge cause severe impact on the pipeline as follows:

(a) Down surge

- During down surge, minimum pressure H_{min} shall be equal to H_o-$h_{srg.}$
- Although down surge always causes a pressure drop, the minimum pressure may or may not be below atmospheric pressure. In a high head system, H_{min} shall still be above pipeline profile, and thus, sub-atmospheric pressures are not encountered. In the small and medium head scheme, sub-atmospheric pressures are likely to occur.
- If the pressure drops to a level of vapor pressure, the liquid vaporises generally at peaks/humps along the pipeline causing a vapour cavity and thus separating water columns on two sides.
- Pressure cannot fall below vapor pressure. Vapor pressure is usually 0.5 to 0.7 m depending on water temperature. Thus, at mean sea level, minimum pressure shall be -10.3 + 0.7, i.e., -9.6 m.
- Separated water columns travel towards the cavity, cause a collapse of the cavity, and creates a shock pressure rise. The shock pressure rises wave travels on both sides and can cause a burst or rupture of the pipeline.
- if sub-atmospheric pressures occur, air may enter the pipeline through flange gaskets or joint rings damaging the seal/gaskets.

(b) Upsurge

During upsurge, maximum pressure shall be equal to $H_o + h_{max}$. If the pressure is above design pressure or field test pressure, a burst or rupture of the pipeline may occur.

6.16.7 Surge Phenomenon due to Single Pump Failure

Even if a single pump of the multi-pump installation fails, sudden velocity reduction does not take place in the pumping main. Hence, no problem is likely to be encountered in the pumping main.

However, due to other running pumps, flow occurs in the delivery of a failed pump from the header in opposite direction to forward flow from the failed pump. This sudden change in velocity from forward to reverse direction causes serious upsurge or overpressure on delivery piping, non-return valve (NRV), and delivery valve. This phenomenon is a common cause of failure of NRV and delivery valve. Unfortunately, no protection measure is discussed in the literature.

To protect the pipeline, it is necessary to design delivery piping and both NRV and delivery valve such that the body of the valve and pipeline are suitable to withstand maximum pressure on surge occurrence. Here, h_{srg} is to be calculated considering velocity in delivery piping and not steady state velocity in pumping main.

6.16.8 Surge Phenomenon in Gravity Main

When any line valve or particularly a control valve at upstream of discharging end is closed, an upsurge or rise in pressure occurs and the wave travels towards the upstream at wave speed c. If the time of closure of the valve exceeds 2L/c where L is the upstream length of gravity main, no problem is encountered. It is, therefore, essential that the time of closure should be gradual, slow, and should be more than 2L/c.

However, pressure rise of moderate magnitude occurs. Normally, a gravity main is designed with a factor of safety above the maximum static head, and adhering to the criteria prevents any possibility of failures.

If the time of closure is less than 2L/c, full surge pressure shall develop. This need to be avoided as gravity main is usually not designed for pressure inclusive of surge pressure.

6.16.9 Guidelines for Design of Pumping Main with and without Surge Protection

a) Without a surge protection device

- The pipeline should be designed to withstand vacuum if encountered and/or maximum design pressure as the sum of operating/working pressure or maximum static head (usually encountered at dip point), whichever is higher, and surge pressure.
- The field test pressure of the pipeline should not be less than the maximum design pressure.
- Shell thickness should be adequate to withstand maximum vacuum with adequate factor of safety.

If the sum of the normal operating pressure and surge pressure exceeds the design pressure, surge protection is essential. Similarly, if a sub-atmospheric pressure occurs or water column separation takes place, surge protection is essential.

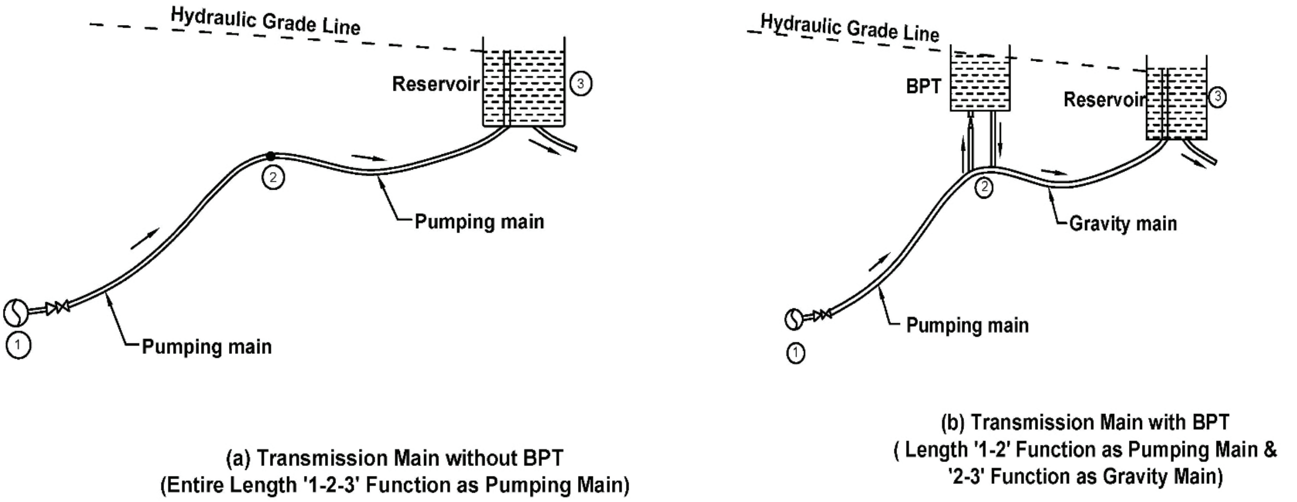
b) With surge protection device

Even if a surge protection device is provided, it is normally advisable to design a pipeline to withstand design pressure inclusive of surge pressure. The dependability of surge protection devices should not be taken 100%.

6.16.10 Strategy for Water Hammer Prevention/Protection of Pumping Main

6.16.10.1 Approaches for Strategy and Available Options

To prevent surge phenomenon by converting the pumping main for maximum length or if feasible full length by introducing break pressure tank (BPT) wherever location for BPT is available at suitable elevation, generally not exceeding 25 m. (Refer Figure 6.26).



(a) Transmission Main without BPT
(Entire Length '1-2-3' Function as Pumping Main)

(b) Transmission Main with BPT
(Length '1-2' Function as Pumping Main &
'2-3' Function as Gravity Main)

Figure 6.26: Conversion of pumping main into partly pumping main and partly gravity main due to BPT

As evident from the discussion, BPT is not a surge protection device. But if provided, it converts downstream pipeline into gravity main, thus practically free from surge pressures. This also reduces the class/rating of the pipeline resulting in a significant saving.

In the pumping system, if a nearby hillock or high ground is available such that HGL is above GL within 20-25 m, BPT can be introduced at the location. Many such pipeline BPT systems are functioning in the country.

For design aspects, refer discussion for BPT in the earlier section of this Chapter.

To provide water hammer protection device either as a single device or multiple complementary devices in series.

Commonly used water hammer protection devices are as under:

(i) Surge tank/surge shaft
(ii) One way surge tank (also called Discharge tank or Feed tank)
(iii) Two-way surge tank
(iv) Air vessel (Air chamber)-Compressor, bladder, and hybrid types
(v) Surge anticipation valve (also called surge suppressor or surge release valve)
(vi) Pressure relief valve
(vii) Zero velocity valve (ZVV)
(viii) Air cushion valve (ACV)
(ix) Standpipe

Unconventional methods or devices which are not commonly used are:

i) Bypass to pumps
ii) Increasing inertia of pump motor set by providing larger flywheel
iii) In-line or intermittent NRV

The essential requirement of NRV in pipeline upstream of location/connection from the surge protection device.

Surge protection devices are provided exclusively for the protection of the pumping main. To make it function effectively, individual delivery pipes, valves, and header need to be isolated by providing NRV/DPCV in the pipeline upstream of the connection tee from the protection device, but downstream of the header. This is also necessary to rely on functioning on common NRV/DPCV instead of individual pump NRVs which may not close simultaneously.

Steps for surge analysis

Surge analysis shall be carried out in the following two steps:

i. Surge analysis for no protection case
ii. Surge analysis with protection device or devices

6.16.10.2 Principles for design and functioning of protection devices

a) Many devices are designed on principle to supply flow into the pipeline as soon as the power fails so as to reduce down surge, prevent sub-atmospheric pressure, and fill the cavity. The upsurge is then correspondingly reduced or may even be entirely eliminated. This is applicable for devices (i), (ii), (iii), and (iv). All the above devices are connected to the pumping main by a tee at the pipeline.

   The devices (iii) and (iv) also control upsurge by throttling reverse flow.

b) The devices (v) and (vi) releases water out from the pipeline; thus, releasing pressure during the upsurge. The devices are fitted on tee branches of the pipeline.
c) The principle behind the design of a zero-velocity valve (ZVV) in (vii), is to arrest forward moving water column when forward velocity is zero and before any return velocity is established. The valve is fitted in-line.
d) The air cushion valve in (viii) as above admits air into the pipeline so as to reduce down surge and fill the cavity. When flow reversal takes place, exit of the air is controlled.

6.16.11 Surge Tank

The device can be selected if HGL at the pumping station is within 20-25 m above the ground level. Refer to Figure 6.27 for the surge tank and surge shaft. The top of the tank is kept above HGL by 1-2 m margins to allow for water level rise during reverse flow or upsurge such as to avoid spilling over. A single connecting pipe from the bottom of the tank is connected to the pumping main with a tee connection. The isolation valve is provided on connecting pipe. The diameter of connecting pipe is usually D/2 where D is the diameter of the pumping main. Only a single pipe connection is required. Preferably the surge tank shall be in RCC construction with dome on top.

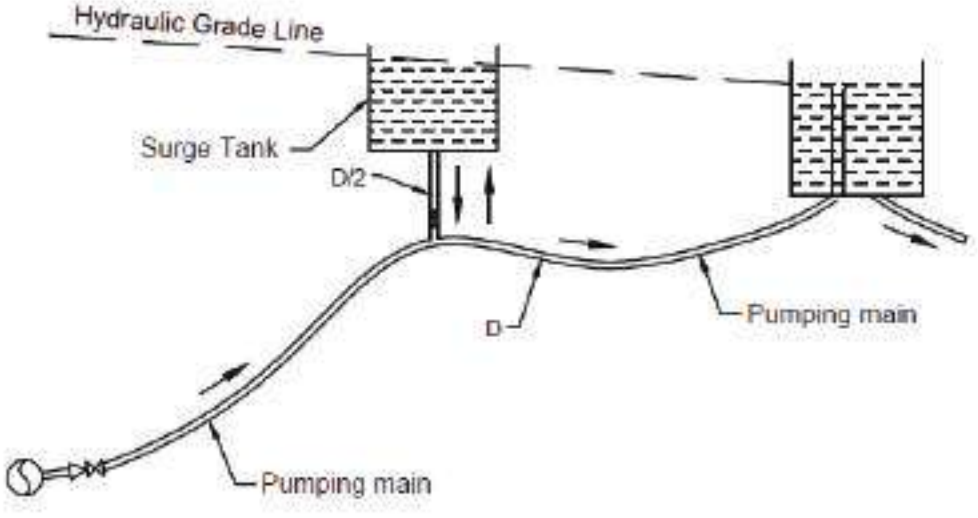


Figure 6.27: Surge Tank

The device is an ideal device amongst all protection devices and should be selected wherever feasible. It is totally without any control, without any moving part or appurtenance, and is maintenance-free.

The design aspect is that during normal operation, the water level (WL) in the tank is at a steady state of WL. The moment power fails and a low-pressure wave starts, flow from the tank/shaft passes to the pumping main reducing the rate of change of flow velocity and thus, reducing down surge and if occurred, filling vapour cavity. Due to outflow, WL in the tank reduces. The cross-sectional area of the tank shall be such as to prevent complete draining; rather, 1-2 m water column needs to be retained. The flow from the device converts an elastic phenomenon into a slow motion phenomenon. During flow reversal, water enters the tank/shaft causing WL to rise.

The amplitude of down surge in the tank/shaft is approximately given by:

$$\text{Amdn} = Vo\sqrt{\frac{Ap\times L}{g\times A_t}} \qquad (6.17)$$

Where Amdn = amplitude of down surge
V_o = steady state velocity
A_p = cross-sectional area of pipeline
A_t = cross-sectional area of a surge tank or surge shaft

The approximate area or diameter calculated as per the above equation shall be rechecked by numerical analysis for unsteady flow.

6.16.12 Surge Shaft

The entire discussion for surge tank including location, connecting pipe, design equation, and method are applicable for surge shaft. Margin above HGL and below lowest water level during down surge shall be 2-3 m. The shaft can be in MS construction. Stays are required to counter the effect of wind as shown in Figure 6.28.

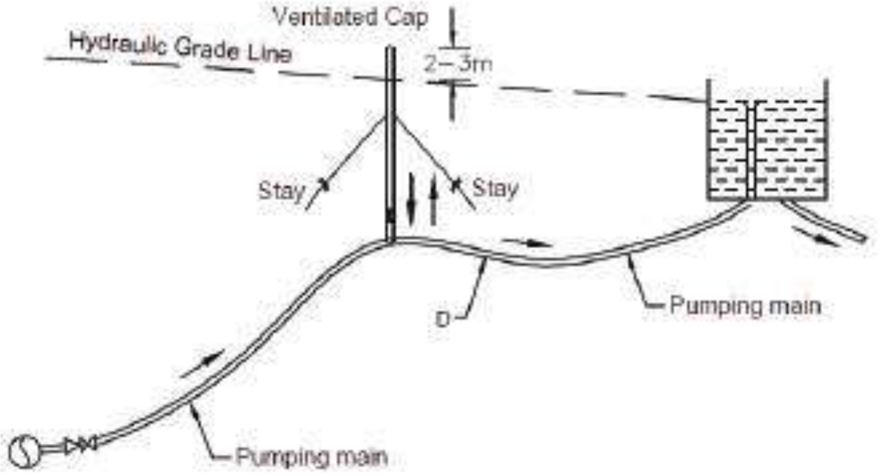


Figure 6.28: Surge Shaft

6.16.13 One-way Surge Tank (Discharge tank / Feed tank)

a) One-way surge tank is located below HGL at maximum up to 25 m height above ground level and connected to pumping main at tee branch. The diameter of connecting pipe is normally D/2 where D is the diameter of the pumping main. Figure 6.29 shows the general arrangement.

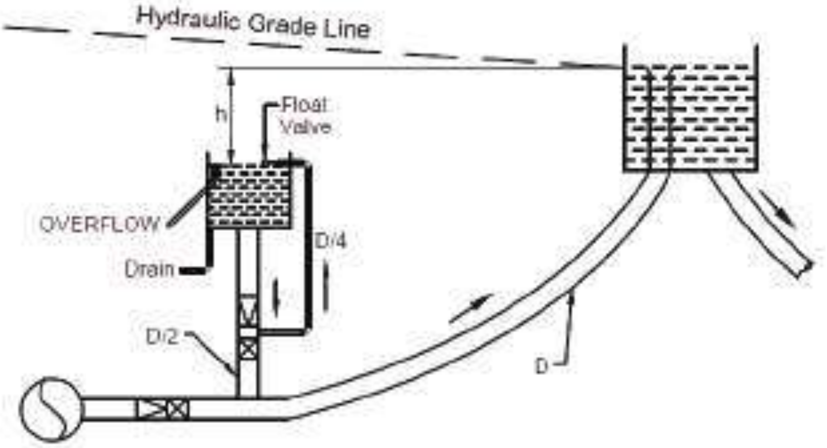


Figure 6.29: One-way Surge Tank

The limitation for the application of the device is that it can function only if $h_{srg} >> h$ (where h is the static head-on tank with reference to discharging end level or subsequent second device).

One-way surge tank functions to reduce down surge; but has no direct capability to reduce to upsurge as under:

When the water level (WL) in the tank is at maximum WL/ TWL, the float valve closes, stopping inflow. The tank during normal operation is at standstill without any inflow or outflow. Following a power failure, velocity in the pipeline starts reducing causing a down surge and dropping pressure in the pipeline. When the transitional surge gradient causes pressure below TWL in one-way surge tank, flow from this tank passes through NRV into the pipeline, thus reducing down surge.

Methods for determining the capacity of a one-way surge tank are given by the following two authors.

i) Professor D. Stephenson in the book ‘Pipeline Design for Water Engineers’ Quantity discharged is given by,

$$\text{Qty} = \frac{A_p \times L \times Vo^2}{2 \times g \times h} \tag{6.18}$$

Where:

Ap = cross-sectional area of pipeline
L = Length of pipeline from tank connection to discharging end or second device
Vo = Steady state velocity
H = static head above one-way surge tank (from tank to the discharging end or next tank)

ii) As per Thorley (1994)

$$\text{Xs} = \frac{L \times Vo^2}{2 \times g \times h_L}$$

$$\text{Ln} = \left(1 + \frac{h_L}{S}\right) \tag{6.19}$$

Where, Xs = distance travelled by a liquid column in coming to rest, Vo = steady state velocity, h_L = friction in length L of pipeline, L = Length from one-way surge tank connection to the pipeline end or the next tank, S = Static head from tank WL to discharging end or next tank

Normally, safety margin of 25%-50% is desirable in the capacity of a one-way surge tank.

As stated above, a one-way surge tank can reduce down surge, but has no direct capability to reduce upsurge. However, if down surge reduces, consequently, upsurge also reduces. Professor Stephenson has illustrated magnitudes in the chart which gives the following conclusions.

- If h/h_{srg} is 0.5, upsurge or pressure rise is zero.
- The device is effective up to h/h_{srg} = 0.8 due to which rise in pressure is restricted to 60% of h_{srg}.
- If h/h_{srg} is less than 0.5, upsurge is usually less than 25% of h_{srg}..

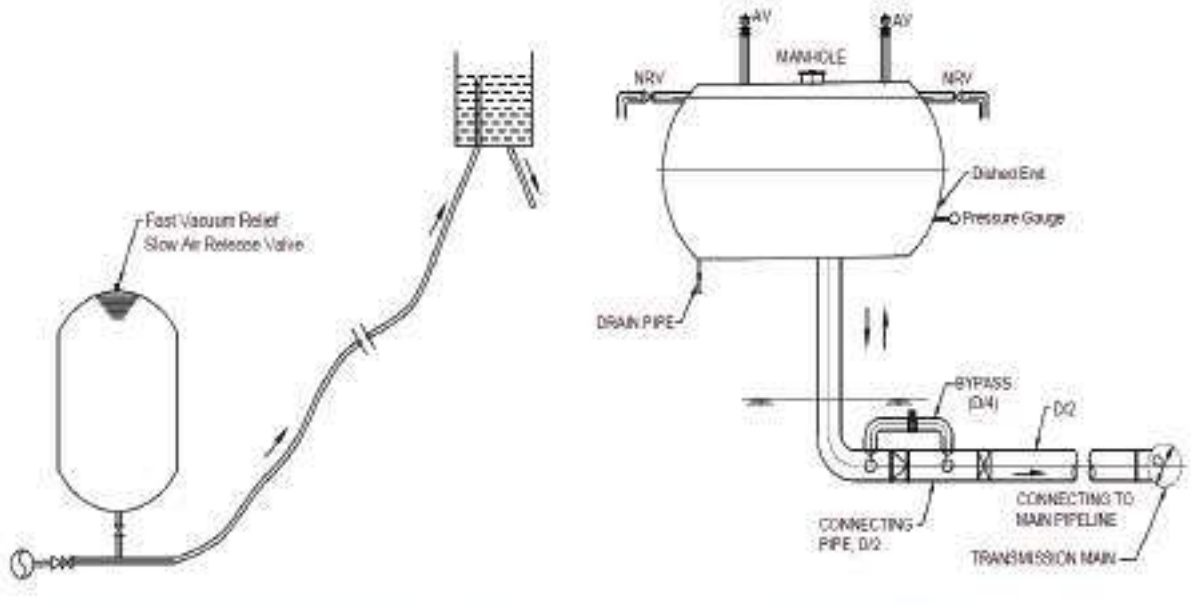
Important aspects of the device are:

- Multiple tanks are required for a pipeline having a number of high peaks.
- A serious demerit of the device is that float valve malfunctions cause spilling of water and thus results in wastage of pumped water.
- It is difficult to monitor spillage due to float valve failure and proper functioning of a one-way surge tank if located away from the pumping station or if unmanned.

6.16.14 Two-way Surge Tank

A two-way surge tank which is an improvisation over features of a one-way surge tank. Please refer Figure 6.30(a) for illustration. As shown, the tank is a closed vessel mounted without a float valve, but with a vacuum breaker and air release valve.

Further, improvisations are attempted in Maharashtra successfully, as shown in Figure 6.30(b). Instead of a vacuum breaker, a non-return valve can be provided on the vessel with direction to open to the tank which admits air into the tank during down surge, thus preventing vacuum in the tank and multiple air valves can be provided to expel air on restarting pumps. An NRV and bypass to NRV are provided.



(a) Two way surge tank (as per book 'Pumping Staton Design')

(b) Two way surge tank (as installed)

Figure 6.30: Two-Way Surge Tank Installation

The limitation for the application of the device is that it can function only if h_{srg} >> h where h is the static head on the tank with reference to a discharging end level or second device.

Like a one-way surge tank, a two-way surge tank is located a maximum of up to 20-25 m above ground level. Following a power failure, velocity in the pipeline starts reducing causing a down surge and drop in pressures in the pipeline. When transitional surge gradient causes pressure below the elevation of the two-way surge tank, flow from the tank passes through NRV into the pipeline, thus reducing down surge. During flow reversal, flow passes into the tank through a smaller bypass pipe, thus throttling the inflow and reducing pressure rise. This also helps in releasing the air at a slower rate.

The charts for one-way surge tanks can also be applicable for two-way surge tanks.

6.16.15 Air Vessel (Air Chamber)

Air vessel is a universal protection device and can be designed for any situation of surge phenomenon both down surge and upsurge and any pipeline with or without peaks. Figure 6.31 shows air vessel installation.

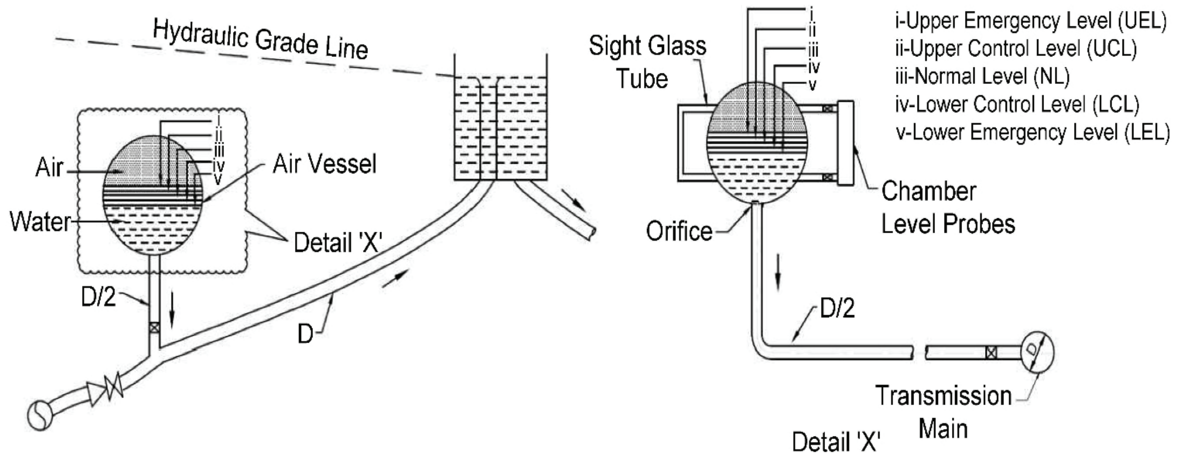


Figure 6.31: Air Vessel Installation

a) Design principle

Design pressure shall be at least equal to the sum of maximum operating pressure and water hammer pressure without any allowance for protection provided. The underlying principle for design pressure is that even if protection is not available for any reason, the air vessel should withstand maximum pressure without any protection and should never fail.

b) Design parameters and functioning

i) Air vessels contain throttling orifice in the outlet. The throttling orifice is designed to achieve minimum head loss during outflow and much higher head loss during inflow.

ii) As the name implies, an air vessel contains water and air on top of the water. The proportion of air and water depends on air vessel design and varies depending on pipeline system characteristics. Important aspects are pipeline parameter and air vessel parameter given by:

Pipeline parameter: $P_p = \frac{c \times V_o}{g \times H_o^*}$ (6.20)

Air vessel parameter: $P_a = \frac{2 \times C_o \times c}{Q_o \times L}$ (6.21)

Where, H_o^* = operating pressure in absolute unit, m, C_o = initial air volume in vessel, m^3 and Q_o = steady state discharge, m^3/s

Other values V_o, g, c and L are as defined previously

iii) During down surge, water flows from the vessel to the pipeline, and the space emptied is occupied by the expansion of air in the vessel as per the following equation,

$$H_o^* C_0^{1.2} = H_{min}^* C^{1.2} \quad (6.22)$$

Where:

H_{min}^* = minimum pressure occurred due to down surge in the absolute unit and C = expanded air volume

(Some investigators use the exponent as 1.3 instead of 1.2. However, 1.2 is more common).

The capacity of the air vessel shall be greater than the expanded air volume to ensure that the air vessel is not fully emptied and air does not enter the pipeline. Normally, about 20%-25% excess volume is kept in design.

Another aspect is that part of air gets dissolved in water causing variation in air-water interface level. Normally, below five levels are kept in interface level bands.

- Normal design interface level at the centre
- Upper interface design level
- Upper emergency interface level (alarm sounds)
- Lower interface design level
- Lower emergency level (alarm sounds)

Normally 25-30 mm spacing is kept in two adjacent levels.

c) Design of air vessel

i) A number of software are available for the design of air vessels. Some widely used software are a) SAP2 developed by the Indian Institute of Science, Bangalore, and b) Surge software Surge: 2018 developed by Kentucky University, USA

ii) The design of an air vessel is quite complex and should be done carefully with the aid of software or charts in books.

d) The necessity of a compressor and air receiver

As discussed, a small portion of air in the vessel gets dissolved in water and therefore, needs to be replenished. Hence, air compressors, generally 1 (working) + 1 (standby) are necessary. To avoid frequent starting and stopping of the air compressor, pressurised air is stored in the air receiver and with level control probes and sensors, the air is fed to the vessel through the air receiver.

Generally, discharge of the air compressor should be adequate to charge the air vessel in a reasonable time of 20-30 minutes.

Air compressor loses pressure building capacity by operation over years. However, the pressure developed by the air compressor should essentially be greater than the operating pressure. In order to allow for the reduction in pressure building capacity, initially specified pressure should be 10%-20% higher than normal operating pressure.

e) Bladder-type air vessel (No compressor is necessary)

In order to avoid the necessity of an air compressor, bladder-type air vessels are developed. The bladder contains compressed air filled corresponding to the required volume C_o at maximum operating pressure. The bladder is a flexible element and stretches to accommodate expanded air volume C. During an upsurge air gets compressed and the bladder can shrink to hold lower air volume on the overpressure.

- The merits of the bladder are that air is not in contact with water. Hence, air volume reduction due to dissolution is prevented and excess capacity air vessels kept 20-25% can be reduced. Further, the bladder vessel requires little or no maintenance.
- The bladder separates water from the air thereby eliminating the need for frequent recharge of compressed air.
- No compressor and no backup generator.
- No additional power requirement for the compressor.
- No mechanical moving parts other than collapsible bladder inside the vessel - little or no downtime.
- Eliminates corrosion of vessel interior as the water is not in contact with air.

- The pipeline may be started almost immediately in case of pump trip following power failure as the bladder gets charged by the line pressure and relatively smaller size vessel does not require much time for charging (gets charged by the time, pumps are running at their rated speed)
- Reduces the high transient pressures generated at the time of normal pump start-up, thereby minimising the start-up times.

However, bladder vessels being relatively new, there is uncertainty about its life and difficulty in monitoring the condition of the bladder. Another demerit is the flexibility of air vessels with compressors for augmented discharge is not available in bladder-type air vessels.

f) Hybrid type air vessel (no compressor, no bladder)

Hybrid type air vessels are provided with a vent on top of the air vessel and a dipping tube. It works similar to a compressor-type air vessel till the pressure drops to atmospheric pressure. The dipping tube controls the closure of the air vent when the tank is filling, and the length of the dipping tube is varied to maintain the desired air volume under normal operating conditions.

g) Connecting pipe from air vessel to pipeline shall generally be D/2 where D is the diameter of the pipeline. Isolating valve (sluice valve/butterfly valve) shall be provided on connecting pipe.

h) Fittings and appurtenances on air vessel

Following fittings and appurtenances are necessary:

i. Safety valve (to release pressure if exceeds beyond preset pressure)

ii. Air release valve (to release air if quantity exceeds)

iii. Drain valve (air vessel is usually installed at 2-3° inclination to facilitate draining)

iv. Level probes and sight glass tube (to monitor air-water interface levels)

v. Pressure gauge

Merits and demerits of air vessel

The merit of the air vessel is that it is the most universal device suitable for any surge phenomenon and any nature of pipeline.

However, the demerits of the device are:

- It is the costliest device compared to other protection devices.
- It requires considerable maintenance, particularly the air compressor, level probes, and sight glass tube.
- If the air-water interface level is not maintained, the effectiveness of the air vessel reduces seriously.
- If the air compressor losses pressure building capacity compared to operating pressure, air feeding to the vessel cannot be done rendering the device inoperative.
- In the case of a bladder-type vessel, monitoring the condition of the bladder is very difficult. If provided, its condition should be checked physically as periodical maintenance by taking shut down and entering into air vessel.

An illustrative example of design of air vessel is enclosed in **Annexure 6.9**.

6.16.16 Surge Anticipation Valve

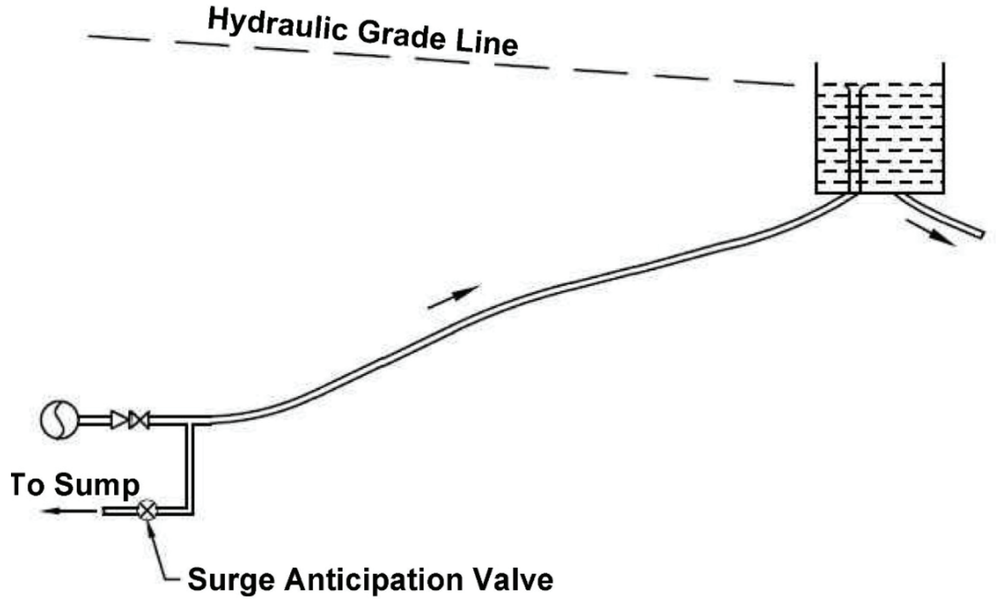


Figure 6.32: General Arrangement of Installation

The valves (Figure 6.32) have actuators that automatically open after sensing a pressure drop or electrical fault following power failure and close at a slower rate after discharging water, thereby releasing high pressure. The water release is throttled to avoid cavitation and erosion due to high outflow velocity. The valve offers protection against pressure rise (upsurge) and has no capability to reduce down surge. The valves are generally installed downstream of the delivery valve and its discharge is conveyed to the suction sump.

Demerits of the valve are:

- Pressure release cannot be controlled within practical accuracy.
- The pilot valve may get clogged due to impurities and suspended matters in raw water.
- If the valve releases water before 2 L/c time, down surge may increase and the possibility of sub-atmospheric pressures or even water column separation is high.
- Cavitation within valve components occurs. Due to high velocity, discharging elements are subjected to erosion.
- Due to time delay in the opening of the valves (about five seconds), they are not suitable for a short pipeline less than 2 km length.
- Preventive maintenance is essential for sensing and operating system.

The merit of the valve is that no control is required for its functioning.

The valves are not advisable for important installations, particularly for raw water pipeline systems.

6.16.17 Spring Loaded Pressure Relief Valve

The valve is set to open and release pressure when the pressure reaches the set value. However, no control can be exercised. The valve being exposed to the atmosphere, the springs in the commercial relief valve get corroded and do not offer desired protection. Its use is restricted to the small pumping system.

(6.16.18 Air Cushion Valve (ACV)

This valve can function only if sub-atmospheric pressure is encountered in the pipeline, i.e., if c V_o/g > operating head at ACV location. Particularly, peak locations are vulnerable.

The principle of this valve is to admit large quantities of air in the pumping main during down surge causing sub-atmospheric pressure occurrence or water column separation. During reverse flow, ACV entraps the air, compresses it with the returning column, and expels the air under controlled pressure by air cushioning.

The valve is mounted to a tee-joint on the pumping main at locations where water column separation is likely to occur.

6.16.19 Zero Velocity Valve (ZVV)

The principle behind the design of the valve is to arrest the forward moving water column at zero momentum i.e., when its velocity is zero and before any return velocity is established. The ZVV is fitted in the line. The valve consists of an outer shell and an inner fixed dome leaving a streamlined annular passage for water. A closing disc is mounted on central and peripheral guide rods and is held in the closed position by one or more springs when there is no flow of water. A bypass connects the upstream and downstream sides of the disc. The springs are so designed that the disc remains in a fully open position for a velocity of water equal to 25% of the designed maximum velocity in the pipeline.

When the forward velocity becomes less than 25% of the maximum, the disc starts closing at the same rate as the velocity of the water. The disc comes to the fully closed position when forward velocity approaches zero magnitudes, water column on the upstream side of the valve is, thus prevented from acquiring a reversed velocity and taking part in creating surge pressures. The bypass valve maintains balanced pressures on the disc and also avoids vacuum on the downstream side of the valve if that column experiences a certain reversal.

Demerits:

(i) Increase in head loss and significant power cost in the normal operation of the flow system.

Serious demerit of the valve in energy is continuously required during operation due to head loss in ZVV as the valve is installed in the line. (All other devices are on tee branches and therefore, do not cause any addition to pipeline head loss.) The head loss is quite significant due to resistance caused by disc held under spring pressures. The head loss is given by:

$$H_L = \frac{KVo^2}{2g}. \tag{6.23}$$

Here K shall be about 2.5 similar for non-return valve. Thus, if V_o = 1.2 m/s with K = 2.5, H_L = 0.184 m, the discharge Q = 0.943 m^3/s, additional power due to ZVV head loss is 2.15 kW and the energy cost per annum (with pump efficiency = 0.85, motor efficiency = 0.93, tariff = Rs.7 per kWh, 22 h/day operation) shall be Rs.1.21 Lakhs per annum which causes major burden on power cost.

(ii) ZVV is not suitable for raw water pipelines. Floating matters can get entangled in the springs due to which closure of ZVV for protection is seriously compromised.

(iii) Considerable periodical maintenance is required. Periodically, any matters entangled in springs need to be removed physically by opening hand holes.

6.16.20 Standpipe

Figure 6.33 shows the general installation arrangement of the standpipe. The top of the standpipe should be 2-3 m above the HGL at the point of tee connection.

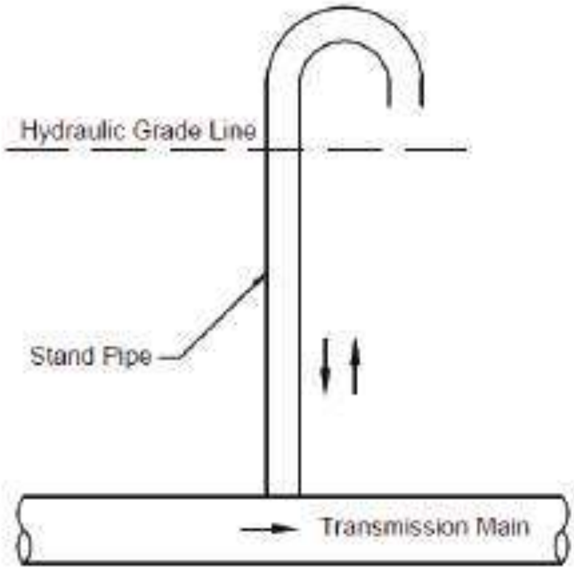


Figure 6.33: General Arrangement of Standpipe

Though the figure shows vertical/upright standpipe, the pipes can also be laid alongside the main pipeline and top terminated 2-3 m above HGL.

The standpipes are generally located at hump/peak which is within 10-12 m of HGL. It prevents sub-atmospheric pressure at tee connection.

6.16.21 Bypass to Low Head Pumps and Booster Pumps

a) This method is applicable for low head pumping systems and also if the pipeline does not have enroute kinks/peaks.

Figure 6.34 shows a typical installation for a low head system. Subsequent to power failure, when the pressure in the pipeline falls below WL in the suction tank, water to the pipeline is fed by the bypass line and consequently reduces the down surge. It is important that the diameter of the bypass line shall be of an adequate size preferably the same as the pipeline diameter so as to cause very low friction loss.

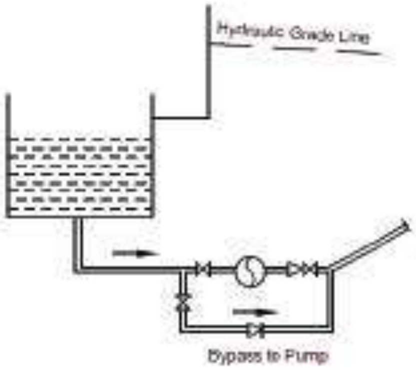


Figure 6.34: A Bypass Line to Low Head Pump

b) This method is very useful for the booster pump system. Figure 6.35 shows the general arrangement. When power failure takes place, the booster pump creates an obstruction to the normal flow resulting in a positive pressure rise on the suction side and pressure drop on the delivery side of the booster. Water flows from the suction side to NRV and further pipelines through the bypass, thus restricting the down surge on the delivery side. The bypass line should preferably be of the same diameter as that of the main pipeline.

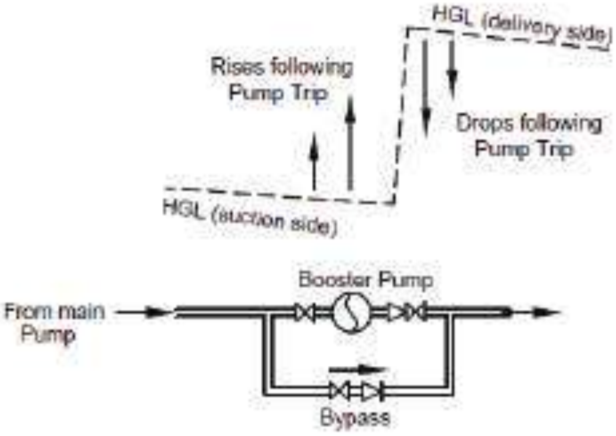


Figure 6.35: Bypass to In-Line Booster Pump Arrangement

6.16.22 Increasing Inertia of Pump Motor Set by Flywheel

The method for calculating pump inertia and motor inertia is described in Chapter 5. Combined inertia of the pump motor set is the sum of pump inertia and motor inertia. A thumb rule is proposed by Professor D. Stephenson in the book ‘Pipeline Design for Water Engineers’ stating that if $\frac{IN^2}{\rho AL\,h_{srg}^2} > 0.01$, the down surge reduces by about 10%.

Here,

I = moment of inertia of pump motor set in kg.m^2

N = r. p.m.

ρ = fluid density, (1000 kg/m^3 for water)

A = pipeline cross-sectional area in m^2

L = length of pipeline, in m

h_{srg} = surge head

Inertia can be increased by adding a flywheel to the motor shaft. However, practically, it has a limitation as under:

- Motor starting current increases requiring up-gradation of switchgear and motor also. Generally, 100% addition in the moment of inertia may increase the motor cost by 20%-25%.
- Flywheel addition is possible only in the case of a horizontal motor. The flywheel cannot be provided on vertical motor.

Thus, adding a flywheel is generally not advisable except for a short pipeline system, where it may be economical.

6.16.23 Suitability and Compatibility of Devices for Series Installation

a) More than one protection device is required for longer pumping mains, depending on the nature and ups-downs of profile and hydraulic characteristics. For large and medium-length pipelines, optimisation can be possible by selecting more than one device to be installed in series. Depending on the length of the pipeline and its profile, even 3-4 devices may be necessary if suitably designed and optimised.

As an example, a very large air vessel was required for over 70 km long pumping main. By introducing a two-way surge tank at enroute hump at 40 km, the capacity of the air vessel was drastically reduced, and the combined cost was half of the cost of a single large air vessel.

It is possible to select devices (Table 6.10) for series installations.

Table 6.10: Devices for series installations

Upstream Choice (Usually at Pumping Station)	Downstream Devices
i) Air vessel	One-way surge tank/two-way surge tank/surge tank/surge shaft/standpipe
ii) Surge tank	One-way surge tank/two-way surge tank/standpipe
iii) One-way surge tank/two-way surge tank	Surge shaft/one-way surge tank/two-way surge tank/standpipe
iv) ZVV and ACV can be used in combination in the pipeline. ACVs are installed at peaks and ZVVs are installed at dip points.	

b) Figure 6.36 illustrates a typical example where the following four protection devices are installed in series.

i) Air vessel adjacent to pumping station
ii) One-way surge tank or two-way surge tank at intermediate peak satisfying condition that $(cV_o/g) > h$ where h is as defined in the section
iii) Surge tank at a pronounced peak where HGL is within 20-25 m above ground level
iv) Standpipe at next pronounced peak where HGL is about 10 m higher than ground level

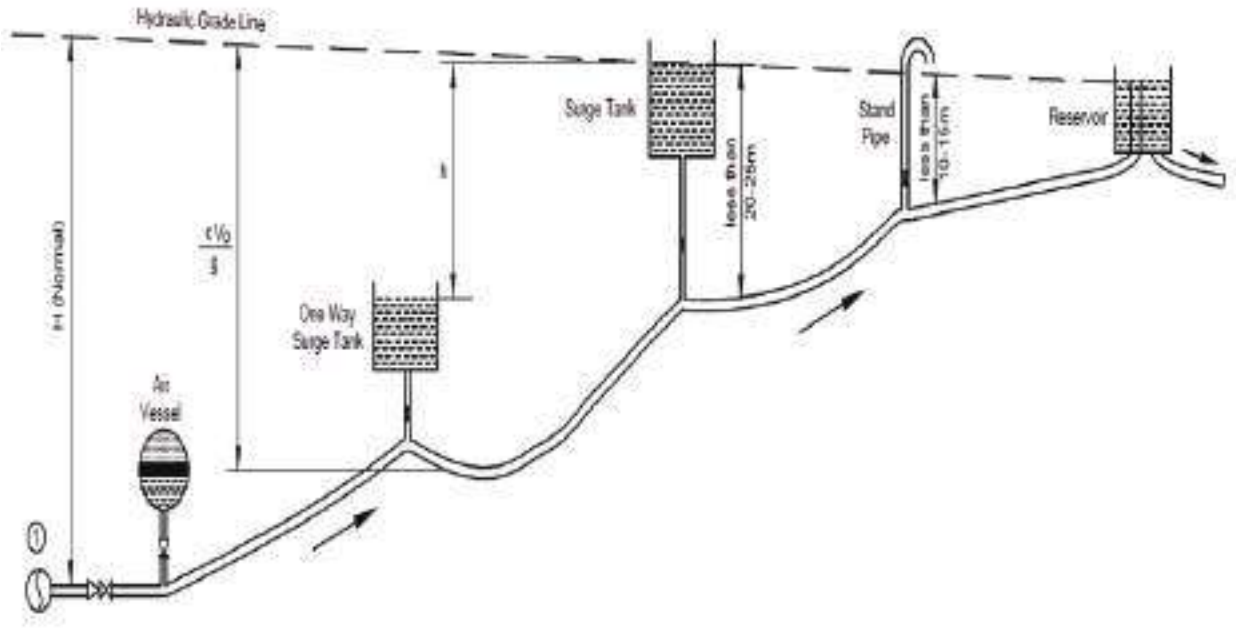


Figure 6.36: Multiple Surge Protection Devices on Very Long Pipeline

c) It is not advisable to provide an air vessel at an intermediate location in combination with other devices at a pumping station due to difficulty in maintaining and controlling interface levels.

6.16.24 In-line Reflux Valve (NRV / DPCV)

Normally, a reflux valve installed on its own in the pipeline will not reduce surge pressures, although it may limit the lateral extent of the shock. In fact, in some situations, indiscriminate positioning of reflux valves in a line could be detrimental to surge pressures. For instance, if a pressure relief valve was installed upstream of the reflux valve, the reflux valve would counteract the effect of the other valve. It may also amplify reflections from branch pipes or collapse vapour pockets.

There are situations where water column separation and the formation of vapour pockets in the pipeline following pump stoppage would be tolerable, provided the vapour pockets did not collapse resulting in surge pressures. Reversal of the water column beyond the vapour pocket could in fact be prevented with an in-line reflux valve at the downstream end of the vapour pocket. The water column would be arrested at its point of minimum momentum, so there would be little head rise.

In locating the reflux valve, allowance should be made for some lateral dispersal of the vapour pocket. The valve should be installed at a suitable dip in the pipeline in order to trap the vapour pocket and to ensure the proper functioning of the valve doors when the water column returns.

A small diameter bypass to the reflux valve should be installed to permit slow refilling of the vapour pocket otherwise overpressures may occur on restarting the pumps. The diameter of the bypass should be of the order of one-tenth of the pipeline diameter. An air release valve should be installed in the pipeline at the peak to release air that would come out of the solution during the period of low pressure.

In-line reflux valves would normally be used in conjunction with a surge tank, one-way surge tank, two-way surge tank, or air vessel where NRV is installed on the upstream side of the tee connection. Following pump shutdown, the tank or vessel would discharge water into the pipe on the downstream side of the reflux valve.

6.16.25 Non-Suitable Devices for Installation in Combination

a) The following combinations are not suitable for series installations:

   (i) Air vessel + ACV (as dependability on ACV is not advisable)
   (ii) Air vessel + ZVV
   (iii) Air vessel + Surge anticipation valve/Pressure relief valve
   (iv) Bypass to pump in combination with other devices

b) Reflux valve/NRV/DPCV should not be generally installed in combination with other devices except for the following positions:

   If intermittent one-way surge tank(s) or two-way surge tank(s) are installed, it is advisable to install RV/NRV/DPCV upstream of connections to the pipeline for second and subsequent tanks to avoid the following situation.

   At some stage after a power failure, second tank (one-way surge tank/two-way surge tank) starts feeding upstream first tank.

6.16.26 Preferred Order for Selection of Devices

On the basis of not only cost but also required maintenance, reliability, and effectiveness of protection, the preferred order for selection can be stated as follows.

Applicable criteria for suitability of the device are restated against each device in Table 6.11 below.

Table 6.11: Preferred Order for Selection of Surge Protection Devices

S. No.	Device/Method	Applicable Conditions	Important Merit/Demerit
i.	**Bypass to pumps**	• $(cV_o/g) > H_o$ • No peak beyond the pumping station • Negligible head loss in bypass	• Very simple, control-free, and maintenance-free • Not suitable for undulating pipeline
ii.	**Surge tank**	• HGL within 20-25 m above ground level	• Very simple, control-free, and maintenance-free; Ideal device
iii.	**Two-way surge tank**	• $(cV_o/g) > h$ • No pronounced peak/hump on downstream pipeline	• Control-free and maintenance-free • Upsurge can be controlled
iv.	**One-way surge tank**	• $(cV_o/g) > h$ • No pronounced peak/hump on downstream pipeline	• Malfunctioning of float valve is a serious handicap in a one-way surge tank causing spilling over and wastage of pumped water and energy • One-way surge tank has no capability for controlling upsurge
v.	**Air vessel**	• Suitable for any pipeline and hydraulic characteristics	• Very costly and needs maintenance

S. No.	Device/Method	Applicable Conditions	Important Merit/Demerit
vi.	**Standpipe**	• HGL within 10-15 m from ground level	• Localised use for avoiding water column separation or sub-atmospheric pressures • No capability to control the upsurge • Spills over during upsurge conditions causing wastage of water and creating an environmental issue
vii.	**Surge anticipation valve (Surge Suppressor/Surge release valve)**	• $(cV_o/g) < H_o$ • Water should be free from floating matters • L > 2000 m	• Not suitable for raw water pipeline • Increases down surge • Need intensive care and maintenance
viii.	**Three stage air valves**	• Suitable for any pipeline	• Usually used in addition to other devices.
viii.	**Pressure relief valve**	• Suitable for short pumping main	• Not dependable
ix.	**ACV**	• $(cV_o/g) > H_o$	• Entire air admitted into the pipeline may not be expelled causing air trapping • Controlling air inflow and air outflow is not within practical accuracy
x.	**ZVV**	-	• Valve is fitted in-line due to which significant head loss takes place causing energy loss and a major burden on operation cost. • No capability for controlling down surge

6.16.27 Surge Phenomenon on Suction Pipes of Pumps

Suction pipes subsystems can be broadly categorised into three types: (a) normal pumping installations where pumps are above/by the side/near suction sump (b) long suction/inlet pipeline from dam or lake at higher elevation (c) In-line booster pump installation.

a) Normal pump installation - Suction pipe short

Nothing of much consequence will happen where the suction line is short irrespective of suction lift or positive suction. The suction sump shall function both as a surge relief outlet as well as a feeding tank and transient surge pressures will dissipate.

b) Long suction pipe/Inlet pipe

Following pump stoppage due to power failure, the long suction pipe/inlet pipe shall be initially subjected to upsurge and subsequently, down surge.

The solution is to control the upsurge by providing a surge tank or surge shaft so as to divert inflow to the protection device, thereby releasing surge pressure and converting surge phenomenon to slow motion phenomenon. Figure 6.37 shows an arrangement of a surge shaft on a long suction line from the dam. The figure also shows an alternative of a surge tank.

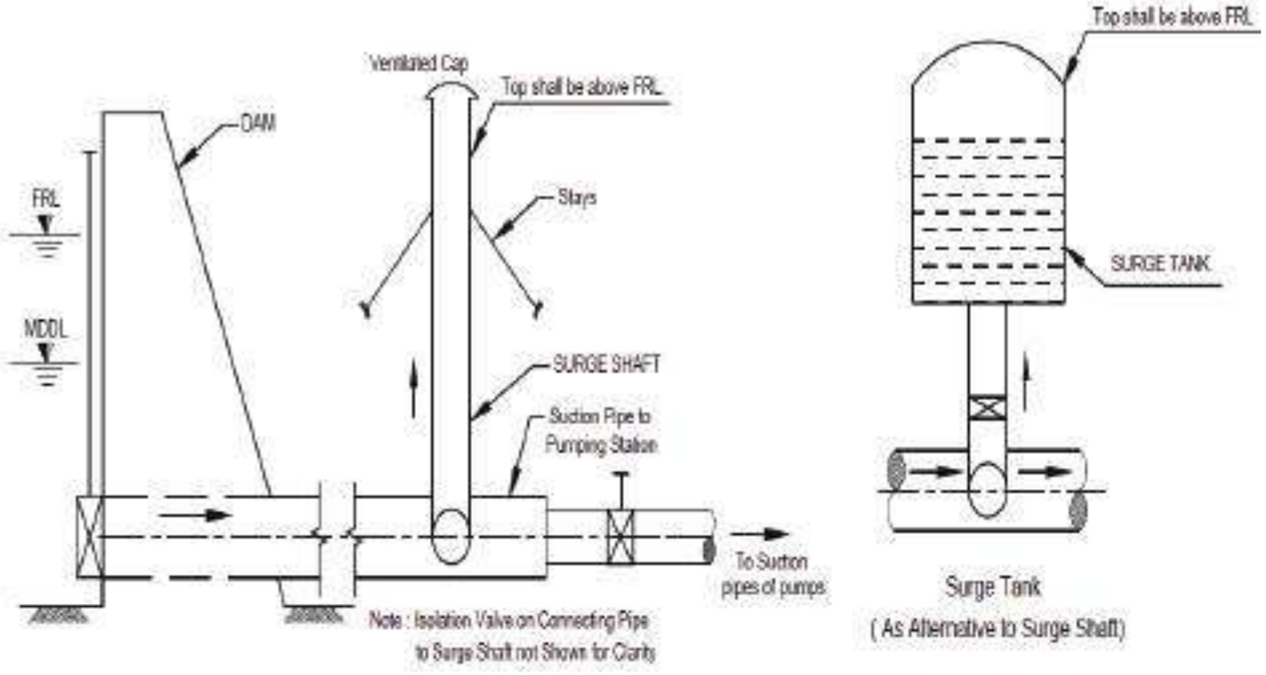


Figure 6.37: Typical Surge Protection Device for Long Suction Pipeline

c) The suction side of the in-line booster pump

Figure 6.37 shows surge occurrence on both the suction side and delivery side of the booster pump. Depending on the pressure on the suction side of the pump, one of the following three types of protection devices can be adopted for protecting the suction pipeline.

i) Surge tank (limit positive pressure on suction side < 20-25 m above ground level
ii) Surge shaft (Same limit as above)
iii) Surge anticipation valve

As for surge protection on the delivery side bypass to a booster, as shown in the figure, it can be provided depending on pipeline characteristics; or if surge pressures are high, other protection devices can be provided after detailed analysis.