20 Water transmission

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20.1 Introduction

Water transmission frequently forms part of small community water supply systems; in that respect they do not differ from large schemes. The water needs to be transported from the source to the treatment plant, if there is one, and onward to the area of distribution. Depending on the topography and local conditions the water may be conveyed through free-flow conduits (figure 20.1), closed conduits (figure 20.2) or a combination of both (figure 20.3). The water conveyance will be either under gravity or by pumping.

Free-flow conduits are generally laid at a uniform slope that closely follows the hydraulic grade line[1]. Examples of such conduits are canals, aqueducts, tunnels or partially filled pipes. If a pipe or tunnel is completely full, the hydraulic gradient and not the slope of the conduit will govern the flow. The hydraulic laws of closed conduit flows, also commonly called pressurised flows, apply in this case. Pressurised pipelines can be laid up- and downhill as needed, as long as they remain at sufficient distance below the hydraulic grade line, i.e. a certain minimum pressure is maintained in the pipe.

Free-flow conduits have a limited application in water supply practice in view of the danger that the water will get contaminated. They are never appropriate for the conveyance of treated water, but may well be used for transmission of raw water.

For community water supply purposes, pressurised pipelines are the most common means of water transmission. Whether for free flow or under pressure, water transmission conduits generally require a considerable capital investment. A careful consideration of all technical options and their costs and discussion with the community groups that will support and manage the system are therefore necessary when selecting the best solution in a particular case.

Routes always need to be checked with community members as well to make use of local knowledge and ensure cultural acceptability (technically desirable routes may, for example, run through a burial site or be unacceptable for other local reasons).

1 Its altitude and the water pressure in it define the piezometric head of each flow cross-section. The hydraulic grade line connects elevations of the piezometric heads and represents the potential energy of the flow. The slope of the hydraulic grade line is called the hydraulic gradient. For free-flow conduits it is the slope of the water surface. For closed conduits the hydraulic grade line slopes according to the head loss.

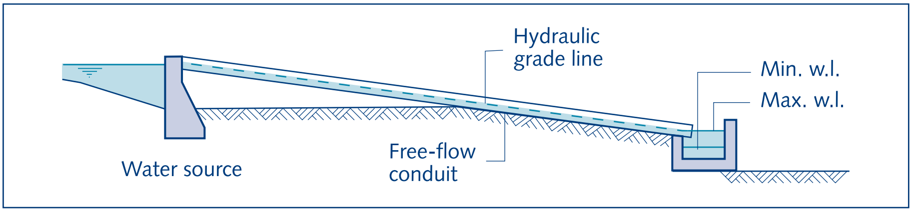


Fig. 20.1. Free-flow conduit

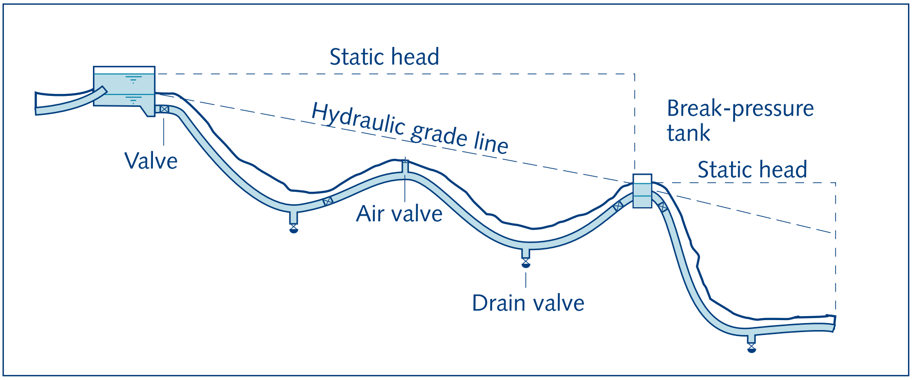


Fig. 20.2. Closed conduit

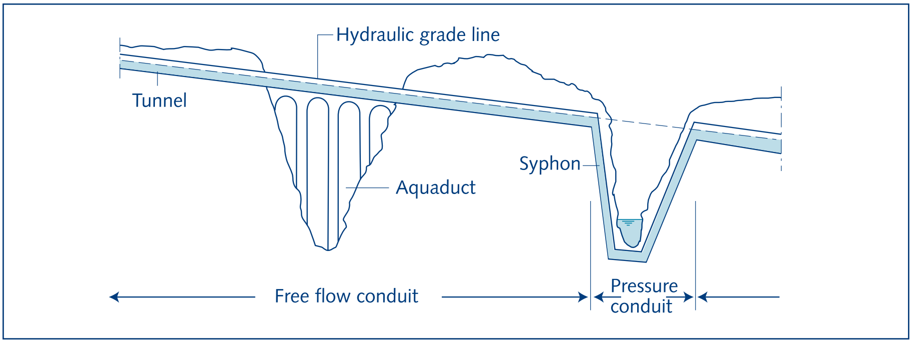


Fig. 20.3. Combined free-flow/closed conduit

20.2 Types of water conduits

Canals

Canals are laid in areas where the required slope of the conduit more or less coincides with the slope of the terrain. Generally they have a trapezoidal cross-section but the rectangular form will be more economical when the canal traverses solid rock. Flow conditions are assumed to be uniform if a canal has the same cross-section dimensions, slope and surface lining throughout its length.

Aqueducts and tunnels

Aqueducts and tunnels are constructed in hilly areas. They should be of such a size that they are approximately three-quarters full at the design flow rate. Tunnels for free-flow water transmission are frequently horseshoe shaped. They are constructed to shorten the overall length of a water transmission route, and to circumvent the need for any conduits traversing uneven terrain. To reduce head losses and infiltration seepage, tunnels are usually lined. However, when constructed in stable rock they require no lining.

Free-flow pipelines

Free-flow pipelines are used for transport of smaller quantities of water than tunnels. Compared with canals and aqueducts they offer better protection from pollution. Due to the free-flow conditions, simple materials may be used for construction. Glazed clay or concrete pipes should be adequate. Similar hydraulic conditions occur as for other free-flow conduits.

Pressurised pipelines

The routing of pressurised pipelines is much less limited by the topography of the area to be traversed, than is the case of canals, aqueducts or free-flow pipelines. A pressure pipeline may run up- and downhill and there is considerable freedom in selecting the pipeline alignment. Nevertheless, such pipelines often follow the topography quite closely, being buried at a similar depth for the length of the route. Also, a routing alongside roads or public ways will be preferred in order to facilitate inspection (for detection of any damage, leakage at pipe joints, faulty valves, etc.) and to provide ready access for maintenance and repair.

20.3 Design parameters

Design flow

The water demand in a distribution area will fluctuate considerably during a day. Usually a service reservoir is provided to accumulate and even out the variation in water demand. The service reservoir is supplied from the transmission main, and is located at a suitable position to be able to supply the distribution system (Fig. 20.4). Again, its site needs to be chosen by the local people, based on technical advice and their own socio-cultural criteria. The transmission main is normally designed for the carrying capacity needed to supply water demand on the maximum consumption day at a constant rate. All hourly variations in the water demand during the day of maximum consumption are then assumed to be evened out by the service reservoir.

The number of hours the transmission main operates each day is another important factor. For a water supply with diesel engine or electric motor-driven pumps, the daily pumping often is limited to 16 hours or less. In such a case, the design flow rate for the transmission main as well as the volume of the service reservoir need to be adjusted accordingly.

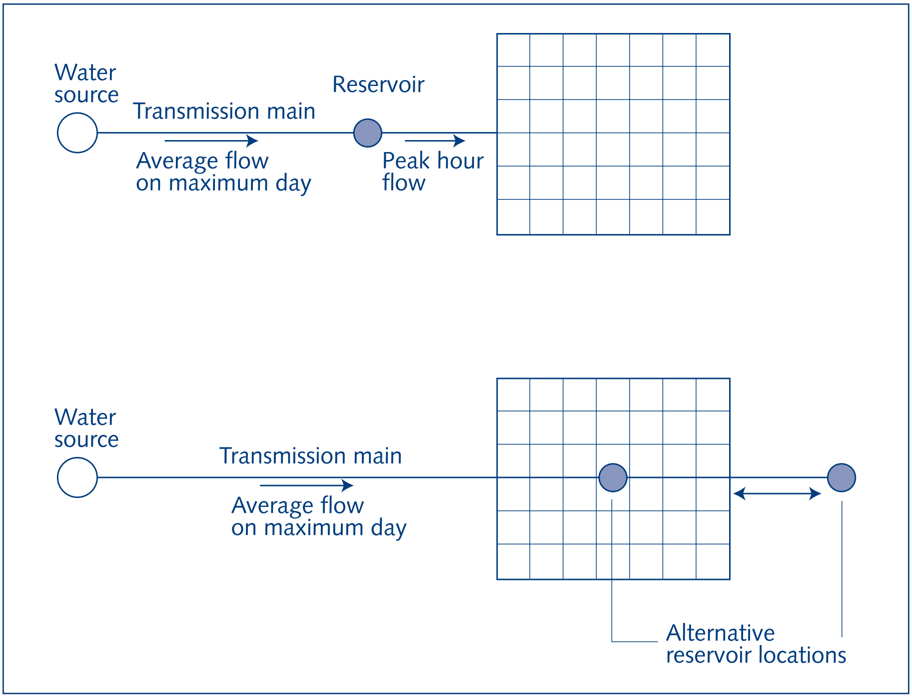


Fig. 20.4. Transmission main and service reservoir (schematic)

Design pressure

Pressure as a design parameter is only relevant for pressurised pipelines. Consumer connections on transmission lines are rare, so the water pressure can be kept low, provided that the hydraulic grade line is positioned above the pipe over its entire length and for all flow rates. A minimum of a few metres water column is also required to prevent intrusion of pollution through damaged parts of the pipe or faulty joints. In fact, nowhere should the operating pressure in the pipeline be less than 4-5 mwc (metres water column).

High pressures in transmission pipes occur as a result of long distances or specific topography. During supply by gravity the maximum pressure does not occur under operating conditions. It is the static pressure when the pipeline is shut (Fig. 20.5). In order to limit the maximum pressure in a pipeline and thus the cost of the pipes, the route can be divided into sections separated by a break-pressure tank. The function of such a tank is to limit the static pressure by providing an open water surface at certain places along the pipeline. The flow from the upstream section can be throttled when necessary.

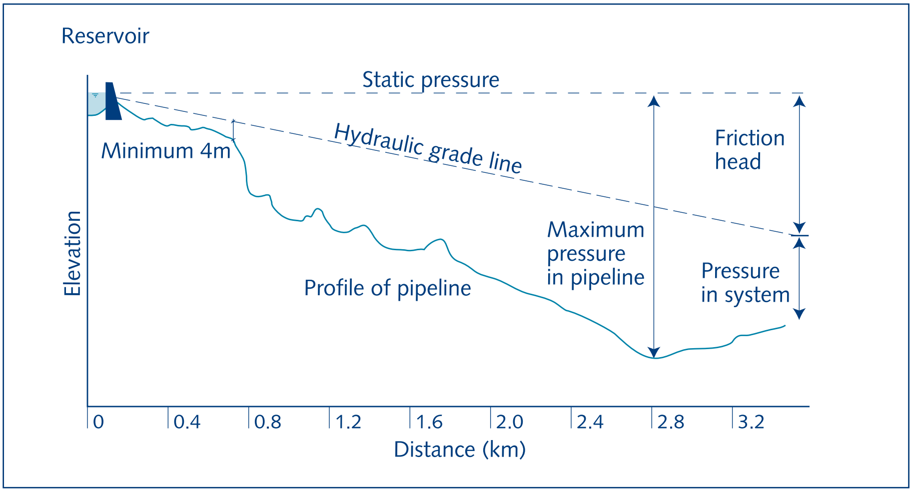


Fig. 20.5. Pressure distribution in gravity transmission mains

If water is to be transported to higher elevations, the maximum pressures will occur in the vicinity of pumping stations (Fig. 20.6). High pressures in the transmission pipe can be avoided in this case by application of multistage pumping along the pipe route.

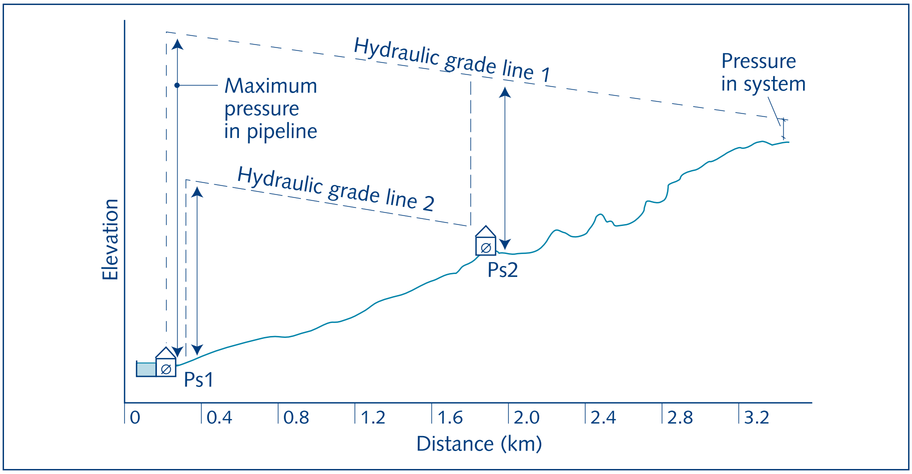


Fig. 20.6. Pressure distribution in pumped transmission mains

Critical pressures may also develop as a result of pressure surge or water hammer in the pipeline. This phenomenon is caused by the instant or too rapid closure of valves, or by sudden pump starts or stops, e.g. due to electricity failure. A longitudinal water wave created in such a way causes over- and under-pressures well above the normal working pressure. This is potentially a very dangerous situation that may result in damage to the pipeline over long distances. Proper prevention includes construction of surge tanks, air vessels or water towers as well as selection of suitable pipe materials that can withstand the highest pressures. Regarding valves, specified minimum shut-off times should be strictly respected. This makes it important how communities choose, train and supervise valve operators and that operators understand, and practise, the proper regulation of the valves.

Design velocity and hydraulic gradient

A velocity range is established for design purposes for two reasons. On the one hand, a certain minimum velocity will be required to prevent water stagnation causing sedimentation and bacteriological growth in the conduits. On the other hand, the maximum velocity will have to be respected in order to control head losses in the system as well as to reduce the effects of water hammer.

The velocity of flows in canals, aqueducts and tunnels usually ranges between 0.4 and 1.0 m/s for unlined conduits, and up to 2 m/s for lined conduits. Flows in pressurised transmission mains have the velocity range between 1 and 2 m/s.

In the case of pressurised pipes, design values may also be set for the hydraulic gradient. This is done primarily to limit the head losses, i.e. to minimise the energy consumption for pumping the water. Common values of the hydraulic gradients for transmission pipes are around 0.005, which means 5 mwc of head loss per km of the pipe length.

20.4 Hydraulic design

Flow Q (m^3/s) through a cross-section A (m^2) is determined as $Q = vA$, where v (m/s) is the mean velocity of the cross-section. Assumptions of 'steady' and 'uniform' flow apply in basic hydraulic calculations for the design of water transmission systems. The flow is steady if the mean velocity of one cross-section remains constant within a certain period of time. If the mean velocity between the two cross-sections is constant at a certain moment, the flow is uniform.

Free-flow conduits

The Strickler formula is widely used for conduits with free-flow conditions. The formula reads:

$$v = K_S R^{2/3} S^{1/2}$$

where:

v = mean water velocity in the cross-section (m/s)
K_S = Strickler coefficient ($m^{1/3}/s$)
R = hydraulic radius (m)
S = hydraulic gradient (m/km)

The Strickler coefficient represents roughness of the conduit. For design purposes, table 20.1 provides indicative values of this coefficient for various types of linings in clean, straight conduits. In practice, these values may differ from one channel section to another and are often subjected to seasonal variations.

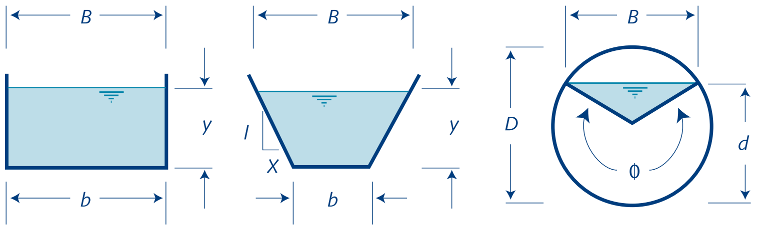
Table 20.1 Indicative values of the Strickler coefficient for various types of linings

Type of Lining	Strickler coefficient K_s ($m^{1/3}/s$)
Planed timber, joints flush	80
Sawn lumber, joints uneven	70
Concrete, trowel finished	80
Masonry	
- Neat cement plaster	70
- Brickwork, good finish	65
- Brickwork, rough	60
Excavated	
- Earth	45
- Gravel	40
- Rock cut, smooth	30
- Rock cut, jagged	25

In a wide range of literature the Strickler coefficient is listed as the Manning coefficient, n in $m^{-1/3}s$, where $n = 1/K_S$. Consequently, the formula is called the Manning formula:

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

The hydraulic radius, $R = A/P$, where A (m^2) is the cross-section area and P (m), the wetted perimeter. The formulas for a few typical cross-sections are listed in figure 20.7. Finally, the hydraulic gradient, S, can be substituted by the slope of the conduit where the assumption of uniform flow conditions is valid.



	Rectangle	Trapezoid	Circle
area, *A*	by	$(b + xy)y$	$\frac{1}{8}(\phi - \sin\phi)D^2$
wetted perimeter, *P*	$b + 2y$	$b + 2y\sqrt{1 + x^2}$	$\frac{1}{2}\phi D$
top width, *B*	b	$b + 2xy$	$\left(\sin\frac{\phi}{2}\right)D$
hydraulic radius, *R*	$\frac{by}{b + 2y}$	$\frac{(b + xy)y}{b + 2y\sqrt{1 + x^2}}$	$\frac{1}{4}\left(1 - \frac{\sin\phi}{\phi}\right)D$

Fig. 20.7. Geometric properties of typical cross-sections

Closed Conduits

The Strickler and Manning formulas are also applicable for closed conduits by introducing the real hydraulic gradient of the flow and the wetted perimeter as the full perimeter of the conduit. Nevertheless, a problem may occur in the selection of the roughness factors for a wide range of pipe materials and flow conditions.
More appropriate formulas for computing the head loss of water flowing through a pressurised pipeline are those of Darcy-Weisbach and Hazen-Williams.

The Darcy-Weisbach formula states:

$$\Delta H = \lambda \frac{L}{D}\frac{v^2}{2g} = \frac{8\lambda L}{\pi^2 g D^5} Q^2 = \frac{\lambda L}{12.1 D^5} Q^2$$

where:

ΔH = head loss (mwc)
L = pipe length (m)
D = pipe diameter (m)
λ = friction factor (-)
v = the mean velocity in the pipe (m/s)
g = gravity (9.81 m/s^2)
Q = flow rate (m^3/s)

Introducing the hydraulic gradient, S = ΔH/L, the formula can be rewritten as:

$$v = \sqrt{\frac{2gDS}{\lambda}}$$

The factor λ is the friction coefficient that can be calculated by the Colebrook-White formula:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{2.51}{\mathrm{Re}\sqrt{\lambda}} + \frac{k}{3.7D}\right]$$

where:

Re = the Reynolds number (-)
k = absolute roughness of the inner pipe wall (mm)
D = pipe diameter (mm)

The Reynolds number indicates the flow regime:

$$\mathrm{Re} = \frac{vD}{\nu}$$

where:

v = the mean velocity in the pipe (m/s)
D = pipe diameter (m)
ν = kinematic viscosity (m^2/s)

Finally, the kinematic viscosity is dependent on the water temperature. For T in °C:

$$\nu = \frac{497 * 10^{-6}}{(T + 42.5)^{1.5}}$$

The Colebrook-White formula is developed for a turbulent flow regime, i.e. Re-values above ± 4000. The common values in practice are much higher, typically in the order of 10^4 and 10^5. If by chance the flow is laminar (Re < 2000), the friction factor λ will be calculated as: λ = 64/Re.

Calculation by the Colebrook-White formula is not straightforward, as the λ-factor appears on both sides of the equation. The alternative formula of Barr can be used instead:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{5.1289}{\mathrm{Re}^{0.89}} + \frac{k}{3.7D}\right]$$

To by-pass somewhat cumbersome computations, the work can also be facilitated by use of the Moody diagram, hydraulic tables or pipe charts (see Chadwick & Morfett, 1996 or Bhave, 1991). These tables/charts, which are produced for certain water temperatures and k-values, show the velocities (flows) for a range of pipe diameters and hydraulic gradients. Any of the three parameters can be determined by fixing the other two.

The common range of k-values is listed in table 20.2 for various pipe materials. For practical calculation these values can be increased depending on the number of years the pipe was in service and the influence of head losses caused by bends, joints, valves, etc.

Table 20.2 Absolute roughness (Bhave, 1991)

Pipe material	k (mm)
Asbestos cement	0.015 - 0.03
Bitumen/Cement lined	0.03
Wrought iron	0.03 - 0.15
Galvanised/Coated cast iron	0.06 - 0.3
Uncoated cast iron	0.15 - 0.6
Ductile iron	0.03 - 0.06
Uncoated steel	0.015 - 0.06
Coated steel	0.03 - 0.15
Concrete	0.06 - 1.5
Plastic, PVC, PE	0.02 - 0.05
Glass fibre	0.06
Brass, cooper, lead	0.003

The Hazen-Williams formula is simpler, although less accurate than the Darcy-Weisbach equation. It states for SI-units:

$$v = 0.355 C_{hw} D^{0.63} S^{0.54}$$

The values of the Hazen-Williams factor, Chw, are listed in table 20.3.

This formula is applicable for a common range of flows and diameters. Its accuracy becomes reduced at lower values of C_{hw} (much below 100), and/or velocities that are appreciably lower or higher than 1 m/s. Also, the Hazen-Williams formula is not dimensionally uniform and if other units are used than SI, it has to be readjusted. Nevertheless, due to its simplicity this formula is still widely used in the USA and in many, predominantly Anglophone, developing countries.

Table 20.3 The Hazen-Williams factors (Bhave, 1991)

Pipe material	C_{hw} D=75 mm	C_{hw} D=150 mm	C_{hw} D=300 mm	C_{hw} D=600 mm	C_{hw} D=1200 mm
Uncoated cast iron	121	125	130	132	134
Coated cast iron	129	133	138	140	141
Uncoated steel	142	145	147	150	150
Coated steel	137	142	145	148	148
Wrought iron	137	143			
Galvanised iron	129	133			
Uncoated asbestos cement	142	145	147	150	
Coated asbestos cement	147	149	150	152	
Concrete, minimum values	69	79	84	90	95
Concrete, maximum values	129	133	138	140	141
Prestressed concrete	147	149	147	150	150
PVC, brass, cooper, lead	142	145	150	152	153
Wavy PVC	147	149	147	150	150
Bitumen/cement lined			150	152	153

Application of the discussed head loss formulas is illustrated in the examples.

Example 1

Determine the capacity of the rectangular concrete canal if the water depth is 0.2 m. The width of the canal is 1.0 m and the slope of the bottom is $S = 1^0/_{00}$.

Solution

From table 20.2, Ks for concrete = 80 $m^{1/3}$/s. Further:

$$R = \frac{A}{P} = \frac{by}{b+2y} = \frac{1*0.2}{1+2*0.2} = 0.1429\,m$$

$$v = K_S R^{2/3} S^{1/2} = 80 * 0.1429^{2/3} * 0.001^{1/3} = 2.19\,m/s$$

$$Q = vby = 2.19 * 1 * 0.2 = 0.437\,m^3/s = 437\,l/s$$

Example 2

Find out the head loss in the concrete transmission pipe, L = 300 m and D = 150 mm, flowing full. The flow rate is 80 m^3/hour and the water temperature is 10°C. Compare the results of the Darcy-Weisbach, Hazen-Williams and Strickler formulas.

Solution

For water temperature of 10°C, the kinematic viscosity:

$$\nu = \frac{497 * 10^{-6}}{(T + 42.5)^{1.5}} = \frac{497 * 10^{-6}}{(10 + 42.5)^{1.5}} = 1.31 * 10^{-6} m^2/s$$

The pipe velocity: $v = \frac{4Q}{D^2\pi} = \frac{4 * 80/3600}{0.15^2 * 3.14} = 1.26\,m/s$

and the Reynolds number: $v = \frac{4Q}{D^2\pi} = \frac{4 * 80/3600}{0.15^2 * 3.14} = 1.26\,m/s$

From table 20.3, the k-value for concrete pipes ranges between 0.06 and 1.5 mm. For k = 0.8 mm, the λ factor from the Barr equation:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{5.1289}{Re^{0.89}} + \frac{k}{3.7D}\right] = -2\log\left[\frac{5.1289}{(1.44 * 10^5)^{0.89}} + \frac{0.8}{3.7 * 150}\right] = 5.60887 \quad ; \quad \lambda = 0.032$$

Finally: $\Delta H = \lambda \frac{L}{D}\frac{v^2}{2g} = 0.032 * \frac{300}{0.15} * \frac{1.26^2}{2 * 9.81} = 5.12\,mwc$

According to table 20.4, the Hazen-Williams factor for ordinary concrete pipe D = 150 mm ranges between 79 and 133. For C_{hw} assumed at 105:

$$v = 0.355 C_{hw} D^{0.63} S^{0.54} \quad ; \quad S = \left(\frac{v}{0.355 C_{hw} D^{0.63}}\right)^{1/0.54} = \left(\frac{1.26}{0.355 * 105 * 0.15^{0.63}}\right)^{1.852} = 0.01722$$

Consequently: $\Delta H = SL = 0.01722 * 300 = 5.17\,mwc$

Finally, for K_S = 85 $m^{1/3}$/s:

$$R = \frac{A}{P} = \frac{\frac{D^2\pi}{4}}{D\pi} = \frac{D}{4} = \frac{0.15}{4} = 0.0375\,m$$

$$v = K_S R^{2/3} S^{1/2} \quad ; \quad S = \left(\frac{v}{K_s R^{2/3}}\right)^2 = \left(\frac{1.26}{85 * 0.0375^{2/3}}\right)^2 = 0.01743$$

$$\Delta H = SL = 0.01743 * 300 = 5.23\,mwc$$

All three formulas show similar results in this case. This can differ more substantially for different choice in roughness parameters. E.g. in case of C_{hw} = 120, the same calculation by the Hazen-Williams formula would yield ΔH = 4.03 mwc while for K_S = 80 $m^{1/3}$/s, the Strickler formula gives ΔH = 5.90 mwc.

In practice, the accuracy of any head loss formula is of less concern than a proper choice of the roughness factor (k, Chw or Ks) for a given surface. Errors in results originate far more frequently from insufficient knowledge about the condition of the conduit, than from a wrong choice of formula.

Example 3

What will be the flow in a 100 mm-diameter pipe to transport water from a small dam to a tank at 600 m distance? The difference between the water surfaces in the two points is 3.60 m. The absolute roughness of the pipe wall is k = 0.25 mm and the water temperature equals 10°C.

Solution

The difference between the water levels indicates the available head loss. Hence, ΔH = 3.60 mwc and S = 3.60/600 = 0.006. From the previous example, $\nu = 1.31 * 10^{-6}$ m^2/s for the water temperature of 10°C. The calculation has to be iterative due to the fact that the velocity (flow) is not known and it influences the Reynolds number, i.e. the flow regime. A common assumption is v = 1 m/s. Further:

$$Re = \frac{1.0 * 0.1}{1.31 * 10^{-6}} = 7.65 * 10^4$$

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{5.1289}{\left(7.65 * 10^4\right)^{0.89}} + \frac{0.25}{3.7 * 100}\right] = 6.0851 \quad ; \quad \lambda = 0.027$$

$$v = \sqrt{\frac{2gDS}{\lambda}} = \sqrt{\frac{2 * 9.81 * 0.1 * 0.006}{0.027}} = 0.66\,m/s$$

The calculated velocity is different from the assumed one of 1 m/s. The procedure has to be repeated starting with this new value. For v = 0.66 m/s, Re = $5.06 * 10^{-4}$, and λ =0.028, which yields v = 0.65 m/s. The difference of 0.01 m/s is considered as acceptable and hence:

$$Q = 0.65 * \frac{0.1^2\pi}{4} = 0.0051\,m^3/s = 5.1\,l/s$$

20.5 Water transmission by pumping

Transmission by pumping is applied in cases when the water has to be transported over large distances and/or to higher elevations. The pumping head is the total head, and comprises the static head plus the friction head loss for the design flow rate. The pump to be selected must be able to provide this head (Fig. 20.9).

The head loss corresponding to the design flow rate can be computed for several pipe diameters using the principles presented in paragraph 20.4. Each combination of the pumping head and corresponding pipe diameter should be capable of supplying the required flow rate over the required distance, and up to the service reservoir. Smaller pipe diameters will require a higher pumping head to overcome the increase in head losses, and the other way round. As a result, one pipe diameter will represent the least-cost choice taking into account the initial costs (capital investment), maintenance costs and the energy costs for pumping. The total cost, capitalised, is the basis for selecting the most economical pipe diameter.

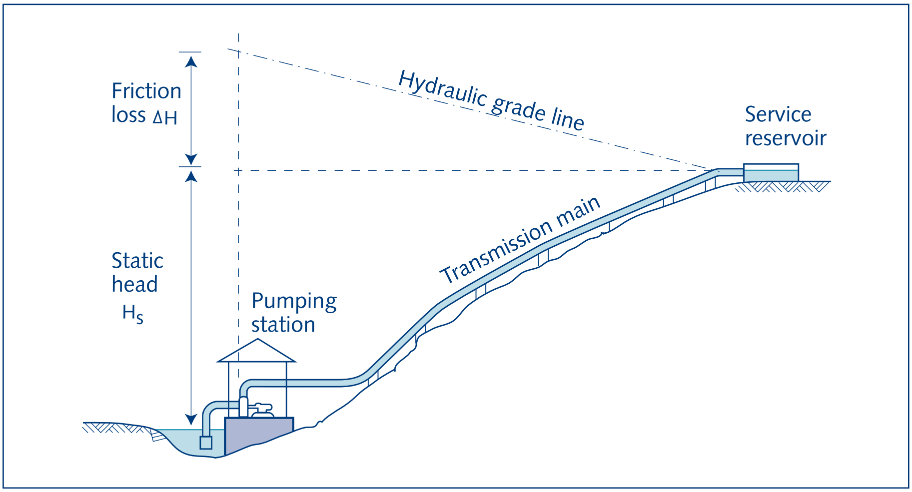


Fig. 20.8. Pumped supply

For this analysis, the calculated costs for different pipe sizes are plotted in a graph of which figure 20.9 shows an example.

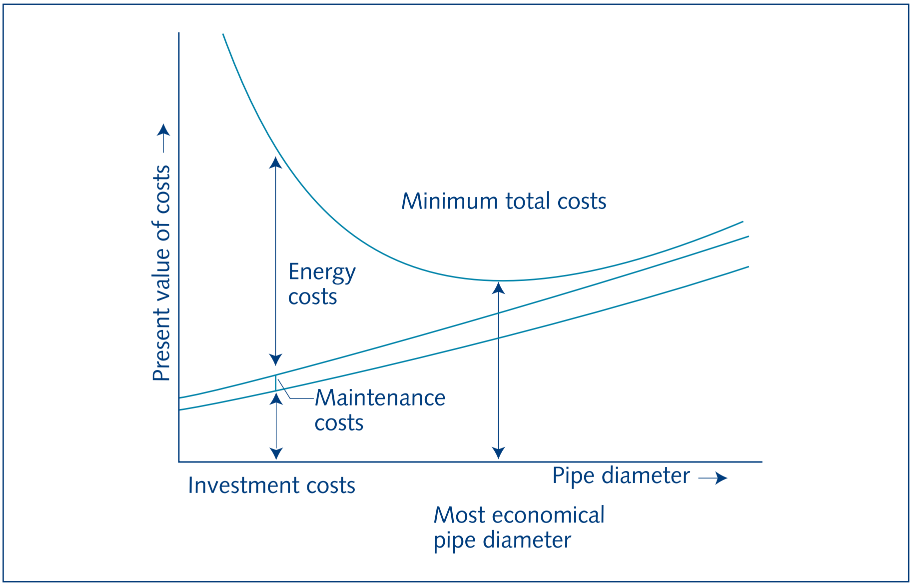


Fig. 20.9. Determination of most economical pipe diameter

The most economical pipe diameter will tend to be large when energy costs are high, unit costs of pipe low, and capital interest rates low. Nevertheless, it should not be forgotten that a larger pipe means lower velocity, i.e. potential water quality problems. As a preliminary estimate, the range of possible most economic diameters can be selected based on velocities around 1 m/s.

Selection of pumps

Various types of pumps have been mentioned in chapter 9: centrifugal, axial-flow, mixed-flow and reciprocating pumps. The choice of pump will generally depend on its duty in terms of pumping head and capacity.

Pumps with rotating parts have either a horizontal or a vertical axis. The choice between these is generally based on the pump-motor drive arrangement and the site conditions. At a site subject to flooding, the motor and any other electrical equipment must be placed above the flood level. Local knowledge is invaluable to identify such risks.

In water transmission for community water supply purposes it is not unusual that a substantial head is required. This implies that the pumps selected frequently are of the centrifugal (radial-flow) type.

Many drinking water pumps are designed to run (almost) continuously throughout a day. In such cases an increase in pump efficiency of a few percent may represent a considerable saving in the running costs over a long period of time. However for rural water supplies, an even more important requirement is that any pump installed should be reliable.

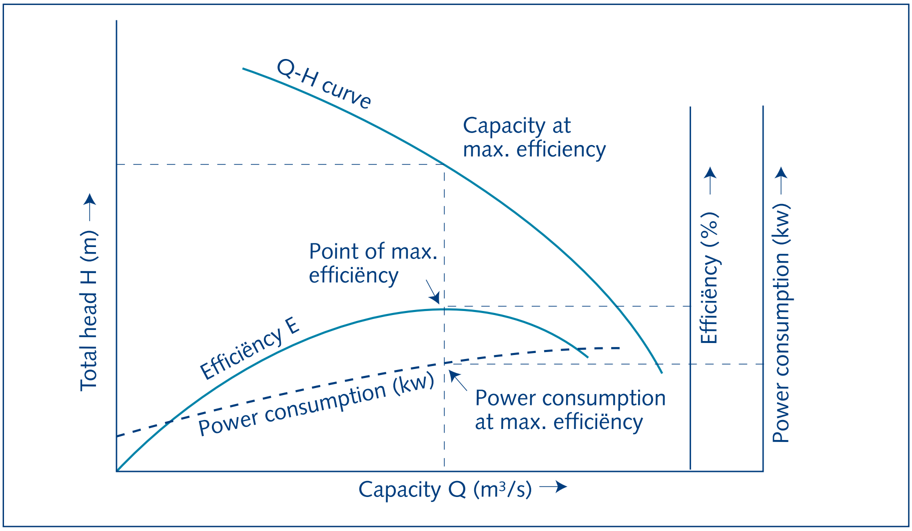


Fig. 20.10. Typical pump characteristics

The head/capacity characteristic of a pump and its efficiency are indicated in catalogues that are supplied by the manufacturers of the pump. Figure 20.10 shows an example.

In practice it is not an easy task to have a pump permanently run at its maximum efficiency because the operating point of the pump is determined by both the pumping head and the capacity, and thus can vary considerably. This applies to some degree for water transmission systems and even more in the case of distribution networks. Efficiencies of small-capacity pumps operating under the conditions of rural areas in developing countries are frequently quite low. A tentative estimate would be as low as 30% for a 0.4-Kilowatt pump, up to 60% for a 4-Kilowatt pump.

Power requirements

The power required for driving a pumping unit can be computed by the following formula:

$$N = \frac{\rho g Q (H_s + SL)}{\eta}$$

where:

N = power required for pumping (Watts)
Q = maximum pumping capacity (m^3/s)
ρ = specific weight of water (kg/m^3)
η = pumping efficiency (-)
H_s = static head (m)
S = hydraulic gradient (m/km)
L = pipe length (m)

Introducing the specific weight in kg/dm^3 gives the result for power in Kilowatts (kW). Assuming $\rho = 1$ kg/dm^3, $g = 10$ m/s^2 and η for small-capacity pumps estimated at 0.5 (50 %), the above formula can be further simplified to:

$$N = 20Q(H_s + SL)$$

This formula gives N in Watts for Q in l/s, either in Kilowatts for Q in m^3/s. The head is in both cases specified in metres of water column (mwc).

Example

For a water supply, pumping is required at rate of 150,000 litres per 12 hours. The static head is 26 m, and the length of the pipeline is 450 m. Determine the power requirement for pumping station, if a PVC pipe, D = 80 mm is used.

Solution

$$Q = \frac{150000}{12 * 3600} = 3.47\,l/s \quad ; \quad v = \frac{0.00347}{\frac{0.08^2 \pi}{4}} = 0.69\,m/s$$

From table 20.4, C_{hw} for PVC of D = 80 mm can be assumed at 147. Further:

$$v = 0.355 C_{hw} D^{0.63} S^{0.54} \quad ; \quad S = \left(\frac{v}{0.355 C_{hw} D^{0.63}}\right)^{1/0.54} = \left(\frac{0.69}{0.355 * 147 * 0.08^{0.63}}\right)^{1.852} = 0.0063$$

$$N = 20Q(H_s + SL) = 20 * 3.47 * (26 + 0.0063 * 450) = 2003\,W \approx 2\,kW$$

Pump Installations

Pumping stations may be of the wet-pit type (with submersible pumps or pumps driven by motors placed above the pump in the sump), or of the dry-pit type (pump installed in a pump room). The wet-pit type has the pumps immersed in the water, and the dry-pit type has the pump in a dry room separated from the water by a wall.

For ease of installation horizontal pumps are sometimes situated above ground level. In that case the pump must be of the self-priming type, which is generally not such a reliable arrangement for rural water supply installations. With too high values of the suction head, a risk of cavitation may occur.

Examples of various types of pump installations are shown in figures 20.11 and 20.12.

20.6 Pipe materials

Pipelines frequently represent a considerable investment and selection of the right type of pipe is important. Pipes are available in various materials, sizes and pressure classes. The most common materials are cast iron (CI), ductile iron (DI), steel, asbestos cement (AC), polyvinyl chloride (PVC) and polyethylene (PE). Galvanised steel (GS) is sometimes selected because of its resilience for situations where subsidence of the pipes is expected. Apart from these, indigenous materials such as bamboo may have limited application.

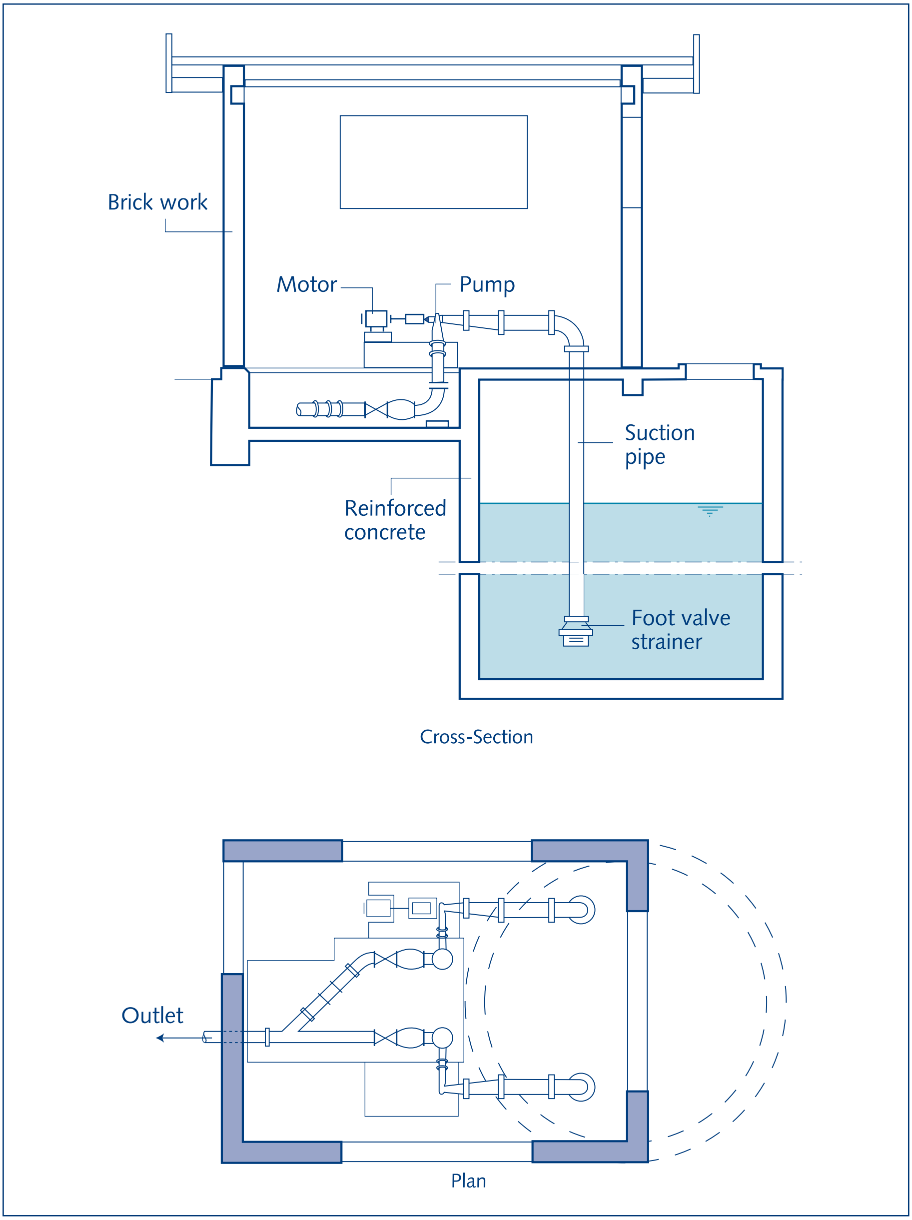


Fig. 20.11. Pumping station with horizontal pumps (self priming)

Factors influencing the choice of pipe material are:

- the cost and local availability of different types of pipe;
- the design pressure in the distribution system;
- the corrosiveness of the water and of the soil in which the pipes are to be laid;
- conditions such as traffic overload, proximity to sewer lines, and crowded residential areas.

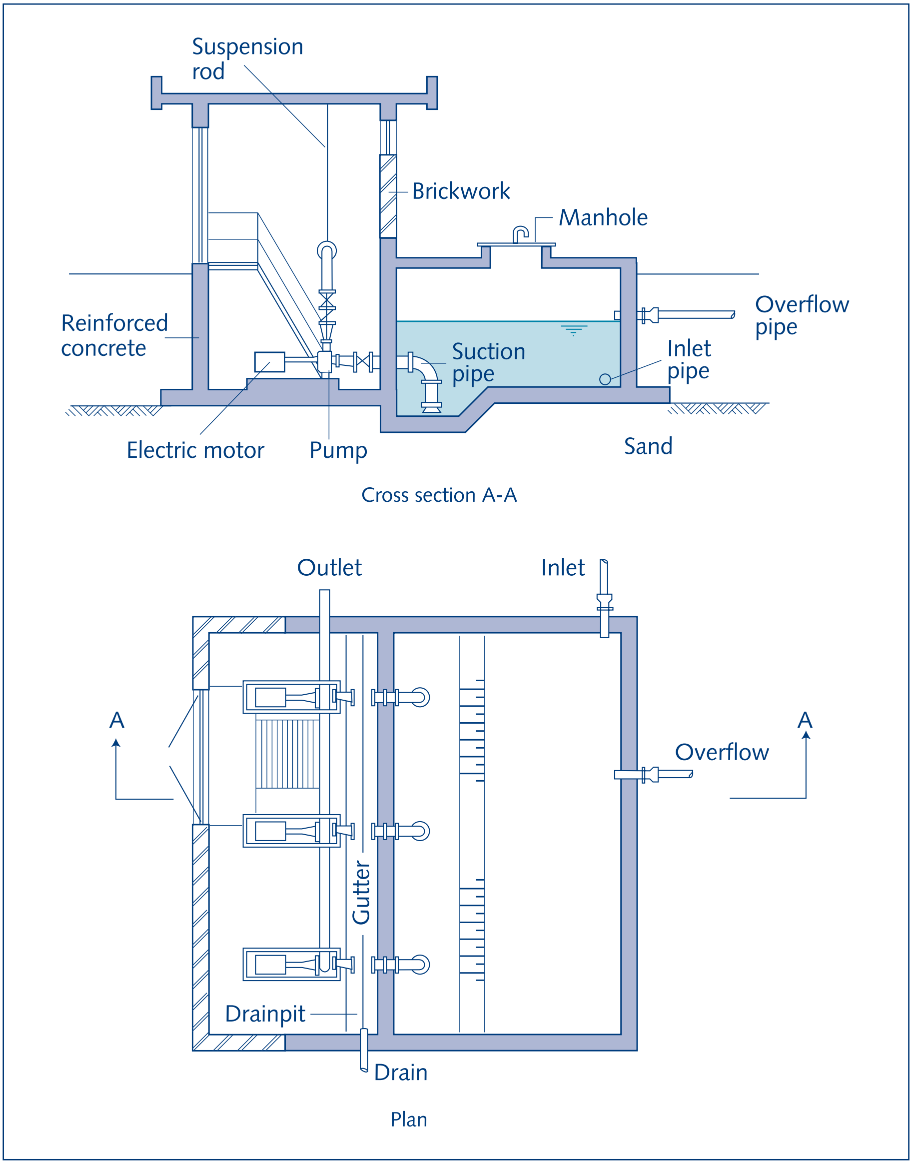


Fig. 20.12. Pumping installation (dry pit)

Although specific conditions will vary from one country to another, the following general observations apply in most cases.

Ductile iron and steel are the strongest pipe materials, making them the best choice when very high operating pressures are to be expected. These pipes are almost impossible to tap without special tools and equipment, which is an advantage in cases where illegal connections pose a real threat for the water company. However, the costs of fittings, valves, etc. increase rapidly for higher pipe pressure classes and it is therefore often advisable to reduce the maximum internal pipe pressure through the provision of a pressure reducing valve or break-pressure tank. A break-pressure tank is generally more reliable than a pressure reducing valve.

In spite of higher investment costs, ductile iron pipes are a better alternative than cast iron pipes because they have a longer service life, are lighter and more flexible and require hardly any maintenance. The pipe is practically corrosion resistant due to coatings applied inside and out.

Compared with metal pipes, asbestos cement pipes are light and easy to handle. Except for soils containing sulphate, these pipes show good corrosion resistance. They are widely used in sizes up to 300 mm, mainly for secondary pipes and for low-pressure mains. Asbestos cement may be less suitable for transmission mains because non-authorised tapping of such mains is possible. Moreover, this pipe material may be subject to scale bursts when tapped without sufficient skill.

The carcinogenic effect of AC materials has been analysed carefully in recent years. Although not dangerous when used to supply drinking water, asbestos fibres can be very harmful when inhaled. AC pipes are slowly being phased-out due to possible hazards during manufacturing and maintenance works. Alternative materials are in this case PVC, PE or DI.

Polyvinyl chloride pipes have the advantage of easy jointing and their corrosion resistance is good. They can be manufactured in several quality classes to meet the selected design pressure. PVC, however, suffers a certain loss in strength when exposed to sunlight for long periods of time and care should be taken to cover the pipes when these are stocked in the open. This is one of the aspects that informed villagers can easily check. Secondly, in case of calamity, PVC pipe breaks along considerable lengths, causing large water losses. Therefore, the pipe should not be laid directly on rocky soil and heavy surface loads are to be avoided. Burying the pipe deeper in the ground can diminish the effect of extreme ambient temperatures. Non-authorised tapping of rigid plastic (PVC) mains is also difficult to prevent.

High-density polyethylene (PE) is a very suitable pipe material for small-diameter mains because it can be supplied in coil. The potential of laying this pipe in longer lengths reduces the number of necessary joints. Particularly in cases where rigid pipe materials would necessitate a considerable number of special parts such as elbows and bends, the flexible PE makes for an ideal pipe material. Polyethylene does not deteriorate when exposed to direct sunlight. Conventional jointing of the PE-pipes may cause leakage and welding is considered to be a better alternative. Furthermore, formation of bio-film in the pipe may be enhanced in some cases.

To summarise, for pipelines of small-diameter (less than 200 mm) PVC and PE may generally be the best alternative unless high working pressures are expected (above 60 mwc). These pipes can also be used for medium- to large-sized pipelines (diameters up to 500-600 mm) where lower pressures can be maintained. Cast iron, ductile iron and steel are generally only used for large-diameter mains and also in cases where very high pressures necessitate their use in small- or mid-range diameter pipes. Due to heavy weight and lower flexibility, CI pipes are becoming less advantageous than DI, despite lower prices. Asbestos cement can be considered only if no other viable alternative exists. Stringent measures that have to be introduced while handling these pipes involve the prevention of the production and inhalation of fibre dust (use of special saws, cutting under wet conditions, protection masks for the workers, etc).

Table 20.4 lists the comparative characteristics of pipe materials for pipelines.

Valves

Apart from the sluice ("gate") valves and non-return valves fitted to the pump outlets in case of a pumped supply, various types of valves and appurtenances are used in the transmission main proper. As the pipeline will normally follow the terrain, provision must be made for the release of trapped air at high points. Air release valves should be provided at all these points on the pipeline and may also be required at intermediate positions along long lengths of even gradient. To avoid under-pressure, air admission valves may also have to be used. These serve to draw air into the pipeline when the internal pressure falls below a certain level. At the lowest points of the pipeline, drain or discharge valves must be installed to facilitate emptying or scouring the pipeline.

In long pipelines sluice valves should be installed to enable sections of the pipeline to be isolated for inspection or repair purposes. Especially when parallel pipes are used it is advantageous to connect them at intervals. In the event of leakage or pipe burst only one section of such an interconnected main needs to be taken out of operation, whereas the other sections and the entire other main can still be used. In this way the capacity of the parallel pipe as such is hardly reduced. It should be mentioned that this advantage is obtained at a cost because each connection between the twin mains requires at least five valves.

Sluice valves perform their function either fully opened or completely closed. For pipe diameters of 350 mm and less, a single valve may be used. For larger diameters a small-diameter bypass with a second valve will be needed because otherwise the closing of the large-diameter valve might prove very difficult. In those cases where the flow of water has to be throttled by means of a valve, butterfly valves should be used. This type of valve may equally be used instead of the sluice valves mentioned above, but the cost is usually somewhat higher.

Table 20.4 Comparison of pipe materials (Smith et al., 2000)

Characteristic	CI	Lined DI	Steel	GS	AC	PVC	PE
Material category	Metal	Metal	Metal	Metal	Concrete	Plastic	Plastic
Int. corrosion resistance	Poor	Good	Poor	Fair	Good	Good	Good
Ext. corrosion resistance	Fair	Moderate	Poor	Fair	Good	Very good	Very good
Cost	Moderate	Moderate	Moderate	Moderate	Low	Low	Low
Weight	High	High	High	Moderate	Moderate	Low	Low
Life expectancy	High	High	High	High	Moderate	Moderate	Moderate
Primary use	T/D*	T/D	T/D	T/D	D	D	S/D
Tapping characteristics	Fair	Good	Good	Good	Fair	Poor	NA
Internal roughness	Moderate to high	Low	Moderate to high	Moderate to high	Low to moderate	Low	Low
Effect on water quality	High	Low	Moderate	Moderate	Low	Moderate	Low
Equipment needs	Moderate	High	Moderate	Moderate	Moderate	Low	Low
Ease of installation	Low to moderate	Low to moderate	Low to moderate	Low to moderate	Moderate	Moderate to high	High
Joint watertightness	Fair	Very good	Very good	Fair	Good	Good	Poor
Pressure range (mwc)	NA	100-250	Varies	NA	70-140	Max.160	Max.140
Diameter range (mm)	NA	80-1600	100-3000	NA	100-1100	100-900	100-1600

* T: transmission, D: distribution, S: service connections

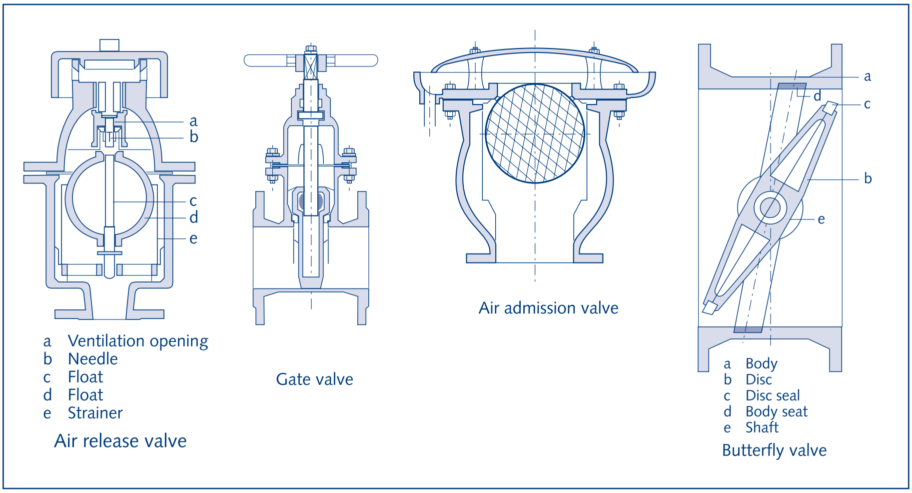


Fig. 20.13. Various types of valves

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