
21 Water distribution

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21 Water distribution

21.1 Introduction

Water distribution systems convey water drawn from the water source or treatment facility, to the point where it is delivered to the users. Unlike the transmission systems discussed in chapter 20, these systems deal with water demand that varies considerably in the course of a day. Water consumption is highest during the hours that water is used for personal hygiene and cleaning, and when food preparation and clothes washing are done. Water use is lowest during the night.

This variation in flow can be dealt with by operating pumps in parallel and/or building balancing storage in the system. For small community water supplies the distribution system with water storage (e.g. a service reservoir) is the preferable option, given that supplies of electricity or diesel to power pumps will usually be unreliable. Although it can be kept simple, construction of such a system may represent a substantial capital investment and the design must be done properly.

Generally, the distribution system of a small community water supply is designed to cater for the domestic and other household water requirements. Stock watering and garden plot irrigation water may also be provided. Service reservoirs accumulate and store water during the night so that it can be supplied during the daytime hours of high water demand.

It is necessary to maintain sufficient pressure in the distribution system to protect it against contamination by the ingress of polluted seepage water. For small community supplies a minimum pressure of 5-10 mwc (metres of water column) should be adequate in most instances.

21.2 Types of distribution systems

There are basically two main layouts of a distribution network:

- Branched configuration (figure 21.1a)
- Looped (or "grid") configuration (figure 21.1b)

Branched networks are predominantly used for small-capacity community supplies delivering the water mostly through public standpipes and having few house connections, if any. Although adequate, having in mind simplicity and acceptable investment costs, branched networks have some disadvantages:

- Low reliability, which affects all users located downstream of any breakdown in the system

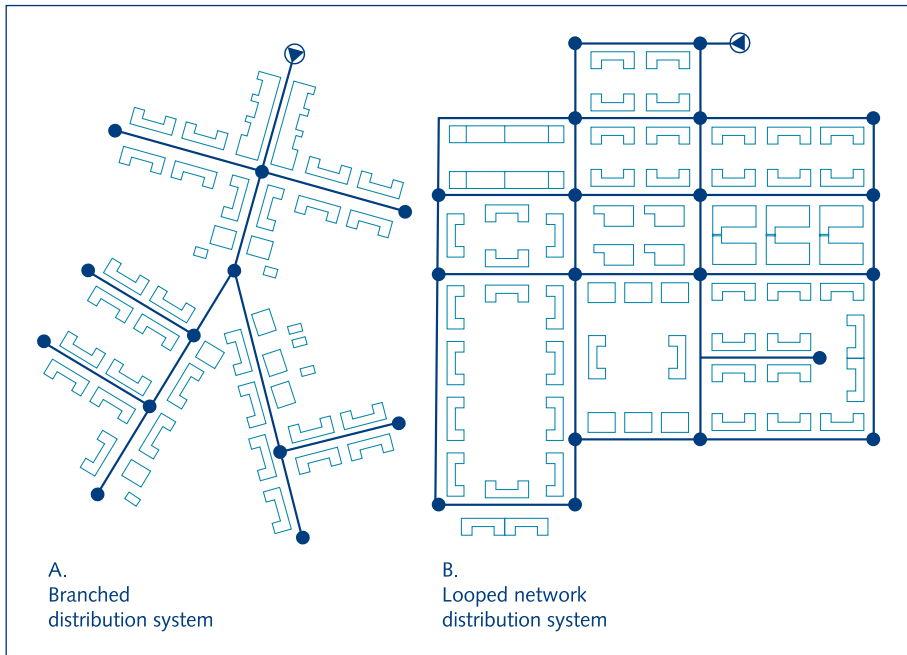


Fig. 21.1. Types of distribution systems

- Danger of contamination caused by the possibility that a large part of network will be without water during irregular situations
- Accumulation of sediments, due to stagnation of the water at the system ends ("dead" ends) occasionally resulting in taste and odour problems
- Fluctuating water demand producing rather large pressure variations

Branched systems are easy to design. The direction of the water flow and the flow rates can readily be determined for all pipes. This is different in looped distribution networks, where consumers can be supplied from more than one direction. Looped networks greatly improve the hydraulics of the distribution system. This is of major importance in the event that one of the mains is out of operation for cleaning or repair.

A looped network usually has a skeleton of secondary mains that can also be in a form of branch, one loop ('ring'), or a number of loops. From there, the water is conveyed towards the distribution pipes and further to the consumers. The secondary mains are connected to one or more loops or *rings*. The network in large (urban) distribution systems will be much more complex (figure 21.2); essentially a combination of loops and branches with lots of interconnected pipes that requires many valves and special parts. To save on equipment costs, over-crossing pipes that are not interconnected may be used but at the cost of reduced reliability (figure 21.3).

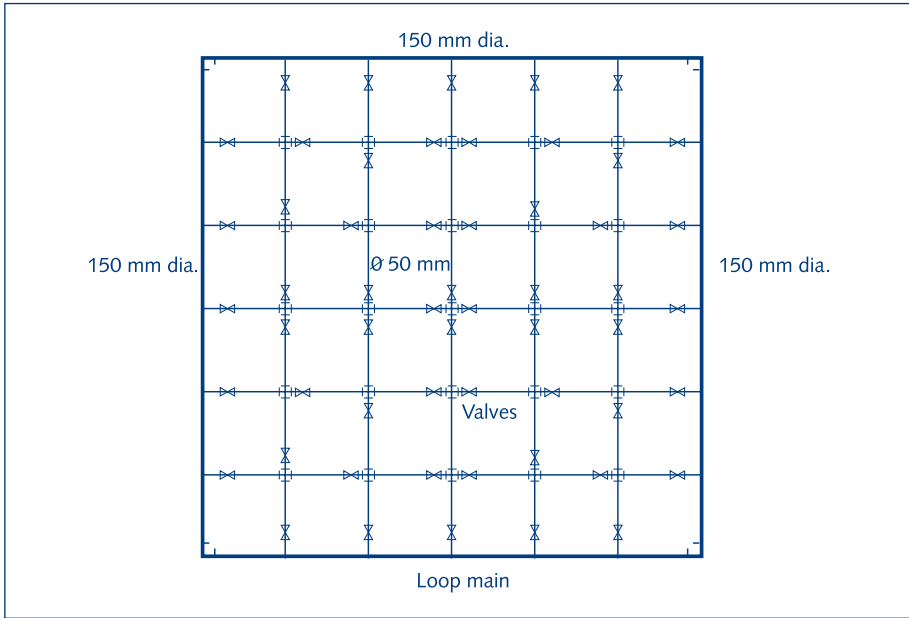


Fig. 21.2. Fully interconnected pipes

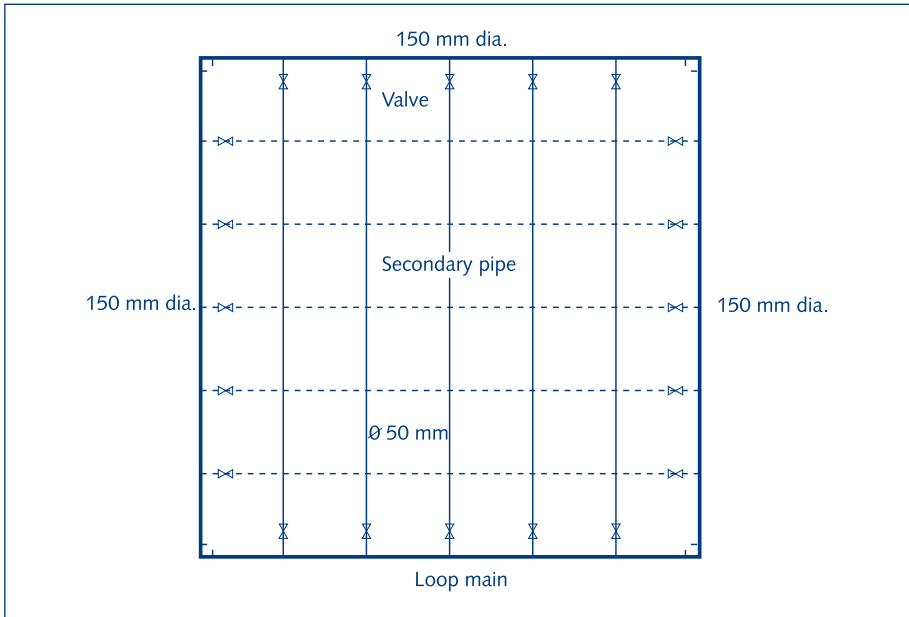


Fig. 21.3. Over-crossing single pipes

Points at which the water is delivered to the users are called service connections. The number and type of service connections has considerable influence on the choice of a network layout.

The following types of service connections may be distinguished:

- House connection
- Yard connection
- Group connection
- Public standpipe

A house connection is a water service pipe connected with in-house plumbing to one or more taps, e.g. in the kitchen and bathroom. Usually 3/8 inch (9 mm) and 1/2 inch (12 mm) taps are used. A typical layout is shown in figure 21.4.

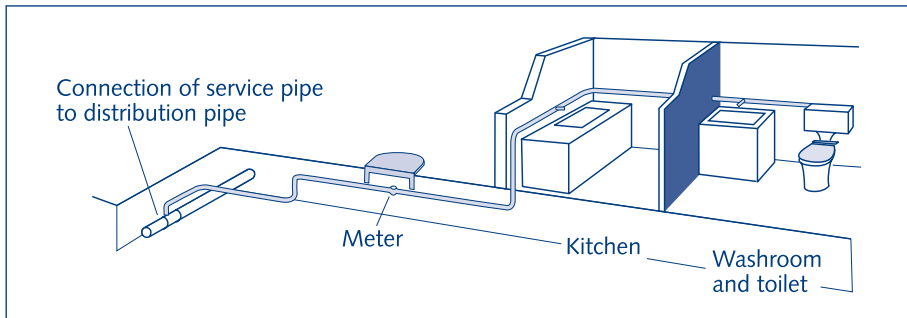


Fig. 21.4. House connection

The service pipe is connected to the distribution main in the street by means of a T-piece (on small-diameter pipes), a special insert piece (*ferrule*) or a saddle (on larger-size secondary pipes). A special insert piece is mostly used for cast iron and ductile iron pipes.

A yard connection is quite similar to a house connection, the only difference being that the tap is placed in the yard outside the house. No in-house piping and fixtures are provided (figure 21.5).

Plastic (polyvinyl chloride or polyethylene), cast iron and galvanised steel pipes are used for both house connections and yard connections.

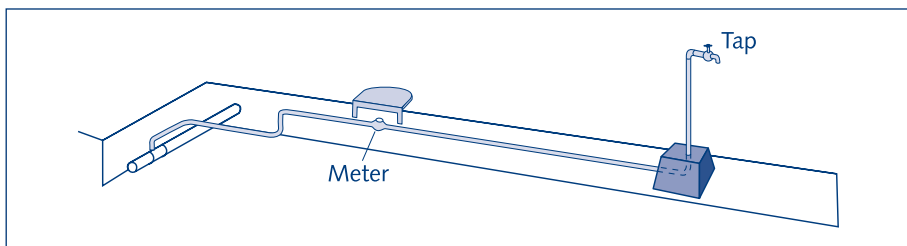


Fig. 21.5. Yard connection

Group connections are outside taps that are shared by a clearly defined group of households, often neighbours. They share the tap and pay the bill together. Each family may pay the same (flat) share or contributions may be weighed according to the estimated volume of water each family consumes. Consumption estimates may be based on indicators such as family size and composition and types of use by the different households. Sometimes the taps can be locked and a local committee holds the key and manages the use and financing.

Public standpipes can have one or more taps. Single-tap and double-tap standpipes are the most common types in rural areas. They are made of brickwork, masonry or concrete, or use wooden poles and similar materials. The design should be done in close consultation with the users (especially women) in order to arrive at an ergonomically optimal solution. Standpipes may have platforms at different levels, making it easy for adults and children to use them with containers of different sizes. Examples are shown in figures 21.6 and 21.7. Cattle watering and/or washing and/or bathing facilities may be constructed nearby. The design and often also the construction is best done in consultation and with participation of the user households, i.e. both men and women. Public taps drawing from a small reservoir (*cistern*) represent an alternative method of water distribution (figure 21.8).

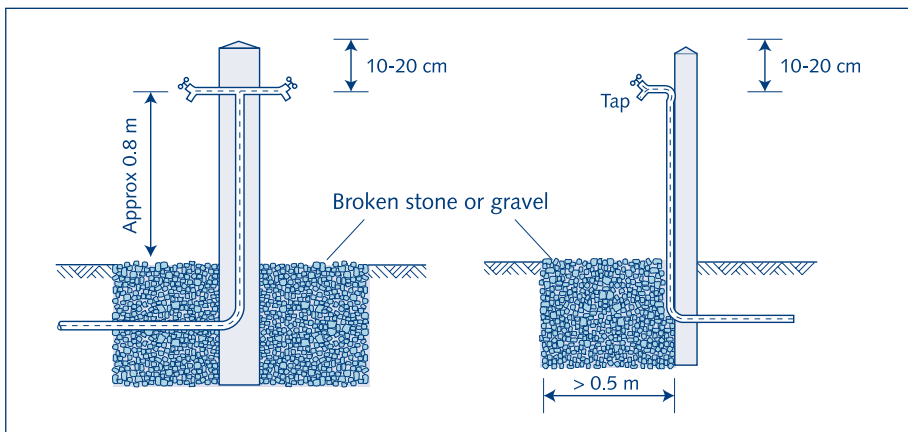


Fig. 21.6. Cross-section of simple standpipes

Each standpipe should be situated at a suitable point within the community area in order to limit the distance the water users have to go to collect their water. The walking distance for the most distant user of a standpipe should, whenever possible, be limited to 200 m; in sparsely populated rural areas 500 m may be acceptable. The required discharge capacity of a standpipe normally is about 14-18 litres/minute at each outlet. A single-tap standpipe should preferably be used by not more than 40-70 people; a multiple-tap standpipe may provide a reasonable service for up to 250-300 persons; in no case should the number of users dependent on one standpipe exceed 500.

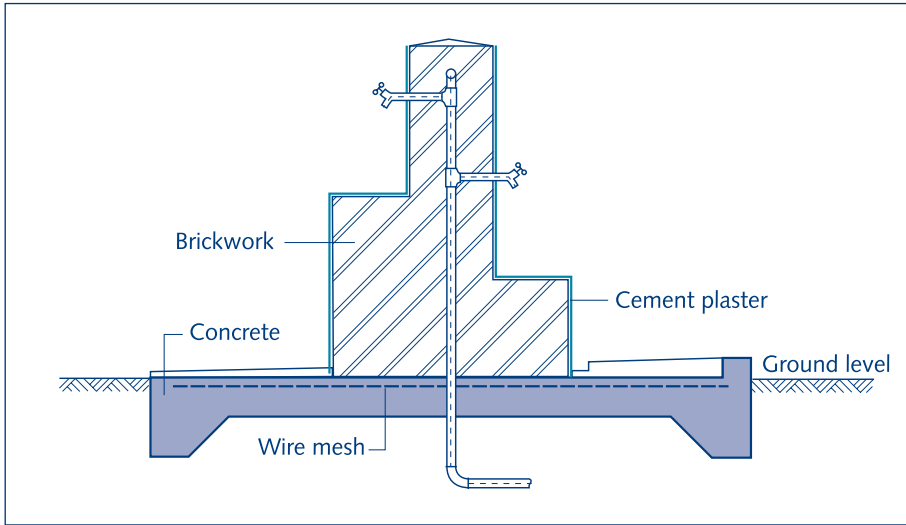


Fig. 21.7. Cross-section of multiple-tap standpipe

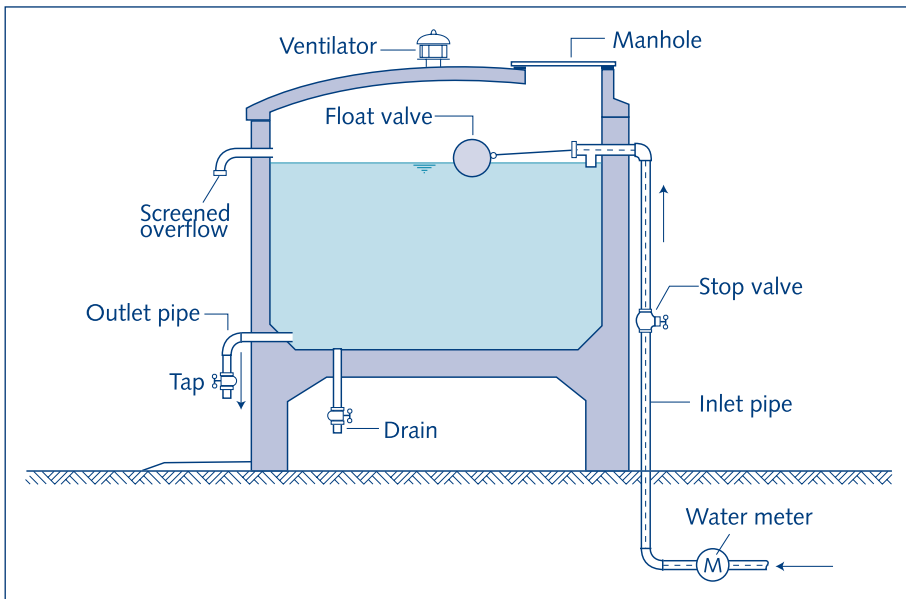


Fig. 21.8. Communal taps supplied from a small reservoir (Cistern)

Public standpipes can operate at a low pressure. Distribution systems that serve only standpipes may therefore use low pressure piping, whereas the pipes for distribution systems with house connections generally have to be of a higher pressure class.

Water collected at a public standpipe will have to be carried home in a container (bucket, jerry can, vessel, pot, etc.). This means that the water that was safe at the moment of drawing may no longer be so at the moment it is used in the house. Water consumption from standpipes generally is not higher than 20-30 litres per person per

day. This consumption increases when other facilities (e.g. for washing/bathing) are added to reduce the amount of water women and children have to carry home. Water use for other purposes than drinking and cooking is likely to be curtailed when the water has to be fetched from a standpipe. Yard and house connections will usually encourage a more generous water use for personal hygiene and cleaning purposes.

Wastage of water from standpipes can be a serious problem, especially when users fail to turn off the taps. Furthermore, poor drainage of spilled water may cause stagnant pools of dirty water with the associated health hazards. It is also not uncommon for the taps to be damaged by the users and pilferage sometimes occurs. These problems occur particularly when designs do not meet the user requirements, i.e. there has not been adequate consultation with the users (women and men) and/or there are no clear management arrangements. One way to cope with these problems is through payment for water consumed, which is a fair and effective way of water demand management. Often, those selling water are women, as they are chosen for their reliability and trustworthiness, their need to be present for work within their own neighbourhood and their suitability as hygiene promoters with other women and children.

In spite of their shortcomings, group connections and public standpipes are really the only practical options for water distribution at minimum cost to a large number of people who cannot afford the much higher costs of house or yard connections. In fact, housing is frequently not suitably constructed to allow the installation of internal plumbing. It would often be impossible for a small community to obtain the substantial capital for a water distribution system with house connections. Also, the costs of adequate disposal of the considerable amounts of wastewater generated by a house-connected water supply service would place an additional heavy financial burden on the community. Consequently, public standpipes have to be provided and the principal concern should be to lessen their inherent shortcomings as much as possible.

To achieve sustainability of this type of service, the management at the public standpipe level needs special attention in terms of organisation, O&M and cost recovery. Chapter 2 gives more specific information about this.

Staged development of distribution systems

It is possible to develop a water distribution system in stages, upgrading it in steps when a community's standard of living improves and funds become available. This is an important point for community consultation, as the initial cost to each household can be limited while they may foresee a future improvement in service level. When designing the distribution system an allowance should be made for its later upgrading. The design engineer has to take into account the higher per capita water demand associated with better household water supply facilities.

The cost of a water distribution system depends mainly on the total length of pipes installed, and less on the diameters of these pipes. Hence, it can be advantageous to design the major components directly for the ultimate capacity. This is even so when initially only part of the distribution system is installed for supplying water at a few standpipes. Thus, for a start, fairly wide-spaced standpipes are provided that probably can be supplied from one or a few mains. An elevated reservoir (or tank) will be very useful to obtain a reliable feeding of water to the distribution system, particularly if the water is taken from the water source by pumping.

In the next stage additional standpipes will be installed in order to reduce the spacing, and thus the distance the water has to be carried by the users. This may mean laying more distribution mains serving the most densely populated clusters in the community. When this basic level of water service has spread throughout the community, the installation of yard taps and house connections may follow. This will probably be concurrent with the provision of yet more standpipes to improve the service to those users dependent on this type of supply. The distribution of public taps is a very sensitive issue and should always be done in a public and accountable manner to avoid certain groups or individuals dominating the choice. Choosing the possible locations in a social map and agreeing on these locations in a public meeting with a quorum of male and female heads of households helps to curb the influence of the “elite”.

21.3 Design considerations

Water demand variation

The daily water demand in a community area will vary during the year due to seasonal climate patterns, the work situation (e.g. harvest time) and other factors, such as cultural or religious occasions. The typical figures for domestic water usage and other water requirements as given in chapter 4 are the averages. The maximum daily demand is usually estimated by adding 10-30% to the average daily demand. Thus, the peak factor for the daily water demand (k_1) is 1.1-1.3.

The hourly variation in domestic water demand during the day is much greater. Generally, two peak periods can be observed: one in the morning and one late in the afternoon (Fig. 21.9). The peak hour demand can be expressed as the average hourly demand multiplied by the hourly peak factor (k_2). For a particular distribution area this factor depends on the size and character of the community served. The hourly peak factor tends to be high for small villages. It is usually lower for larger communities and small towns. Where roof tanks or other water storage vessels are common, the hourly peak factor will be further reduced. Usually, the factor k_2 is chosen in the 1.5-2.5 range. Where a pipe connection is designed to supply a small group of consumers, a higher value should be adopted because of the effects of instantaneous demand.

A water distribution system is typically designed to cater for the maximum hourly demand. This peak demand may then be computed as $k_1 \times k_2 \times$ average hourly demand.

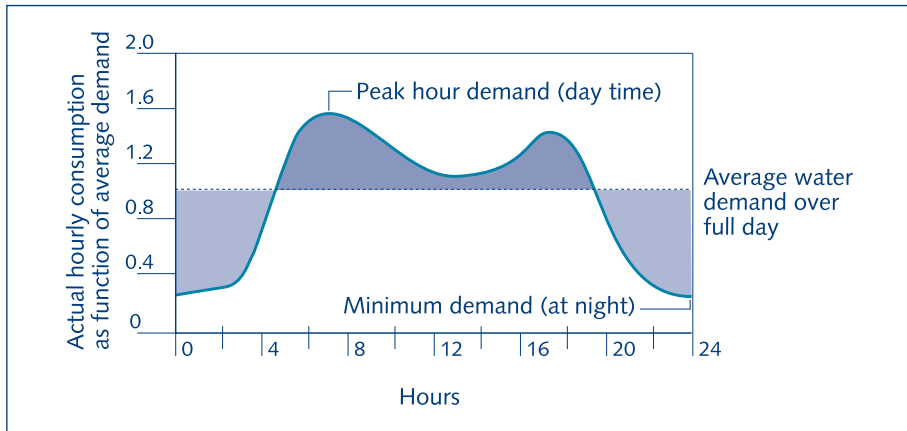


Fig. 21.9. Variation of domestic water demand during the day

Example

For a particular distribution area the average daily water demand is estimated (using the design figures given below) at 500,000 litres per day.

$$\begin{aligned} Q_{\text{average day}} &= 500,000 \text{ l/d} \\ Q_{\text{peak day}} &= 1.2 \times 500,000 = 600,000 \text{ l/d} \\ Q_{\text{average hour on peak day}} &= 600,000 / 24 = 25,000 \text{ l/h} \\ Q_{\text{peak hour}} &= 1.8 \times 25,000 = 45,000 \text{ l/h} = 12.5 \text{ l/s} \end{aligned}$$

The above calculation reflects the situation of an existing system where the average daily demand is obtained from the field, usually by measuring at supply points. Hence, the registered demand and therefore the peak factors will include leakage.

While developing a new system or extension, the starting point can also be some specific consumption that does not necessarily include leakage. In that case the leakage percentage has to be added in the following way:

$$Q_{\text{peak hour}} = \frac{Q_{\text{average day}}}{f} \left(k_1 k_2 + \frac{l}{100 - l} \right)$$

Factor, f , in the equation is a unit conversion factor while l represents the leakage percentage of the total quantity supplied to the system.

It is common to assume that 'demand = consumption + leakage'. If in the above example $Q_{\text{average day}}$ would represent pure consumption, the peak hour demand with leakage of, say, 20% becomes:

$$Q_{\text{peak hour}} = \frac{500,000}{24 \times 3600} \left(1.2 \times 1.8 + \frac{20}{100 - 20} \right) = 14.0 \text{ l/s}$$

Balancing storage

Without storage of water in the distribution area the source of supply and the water treatment plant would have to be able to follow all fluctuations in the water demand of the community served. This is generally not economical, and sometimes not even technically feasible. The design capacities of the various components of a water supply system are usually chosen as indicated in figure 21.11.

In summary:

System component	Design capacity
Water source	Peak day water demand
Raw water main	Peak day water demand
Treatment plant	Peak day water demand
Transmission main	Peak day water demand
Distribution system	Peak hour water demand

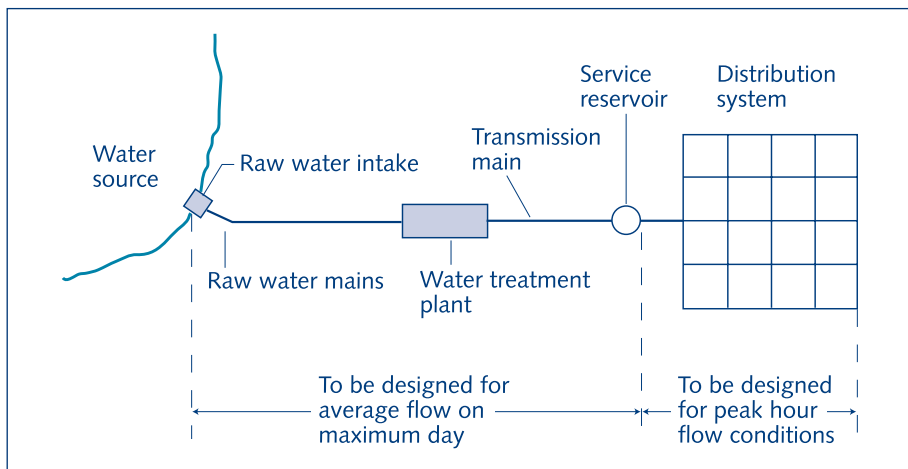


Fig. 21.10. Design capacities for water supply system components

The service reservoir is provided to balance the (constant) supply rate from the water source/treatment plant with the fluctuating water demand in the distribution area. The storage volume should be large enough to accommodate the cumulative differences between water supply and demand.

The required storage volume can be determined as follows. The estimated hourly water demand (example given in figure 21.9) is expressed as a percentage of the total demand over the peak day and plotted in a cumulative water demand curve (Fig. 21.11). The constant supply rate is then drawn in the same diagram as a straight line¹.

¹ In the example the supply operates at a constant rate. If the supply rate is not constant, a broken line will represent the cumulative quantity of water supplied.

The required volume of storage can now be read from the graph. For a constant-rate supply, 24 hours a day, the required storage is represented by A-A' plus B-B', which is about 28% of the total peak day demand. This figure is directly proportional to the two shaded areas in figure 21.9. The balancing volume available at the beginning of the day should equal the B-B' percentage of the peak day demand. In that way, the tank will be full at the moment the A-A' percentage has been added to it and empty i.e. at the reserve volume when the B-B' deficit has been reached. These two moments occur, for example, in figure 21.11 after four hours and 20 hours, respectively. Based on the shape of the tank the expected water level variation in the tank can also be determined. If the supply capacity is so high that the daily demand can be met with 12 hours pumping a day, the required storage is found to be C-C' plus D-D', in this case about 22% of the total peak day demand. However, if the same pumping takes place overnight or in intervals (in order to reduce the load on the electricity network, i.e. save by pumping at cheaper tariff), the required balancing volume will have to be much larger. For example for pumping between 6 p.m. and 6 a.m., C'-C'' + D'-D'' \approx 76%.

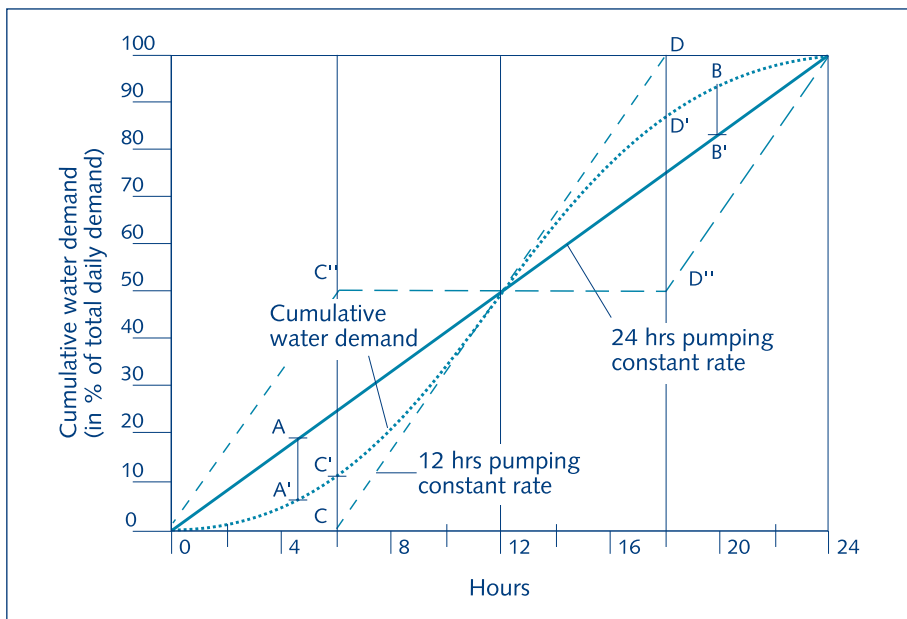


Fig. 21.11. Graphical determination of required storage volume (service reservoir)

A service reservoir with a storage volume of 25-40% of the peak day water demand should generally be adequate. Yet, a larger reservoir may be called for in situations where any interruption of the water supply would be particularly critical.

Any reservoir should be situated at a higher elevation than the distribution area, but as close to it as possible. If such a site is available only at some distance, the reservoir has to be placed there. Figure 21.12 shows two possible arrangements.

In flat areas where no suitable hill sites or other high points for ground reservoirs are available, elevated tanks have to be used. In theory such storage should have the same balancing volume as the ground reservoir. In practice, however, these are usually costly solutions. Alternatively, more water is pumped directly into the network, in combination with an elevated tank of much smaller size (Fig. 21.13). These tanks, commonly called water towers, are less than 1000 m³ and serve for minor balancing of demand variations. In essence, their prime purpose is to stabilise pumping station operation and pressures in the system. They also serve as a protection against water hammer or as a temporary buffer in cases of power failure before a diesel generator can be switched on (if one is available).

A set-up with a ground reservoir on the suction side of the pumping station can also be used (Fig. 21.14). It allows for stable production at the source and also creates a buffer in case this production is temporarily terminated. This, however, is too complex an arrangement for small community supplies.

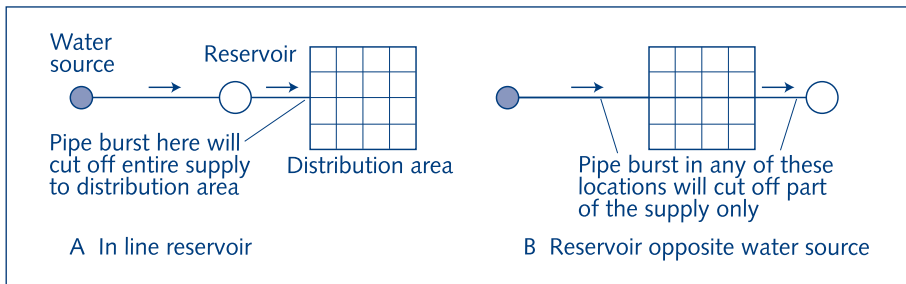


Fig. 21.12. Reservoir siting

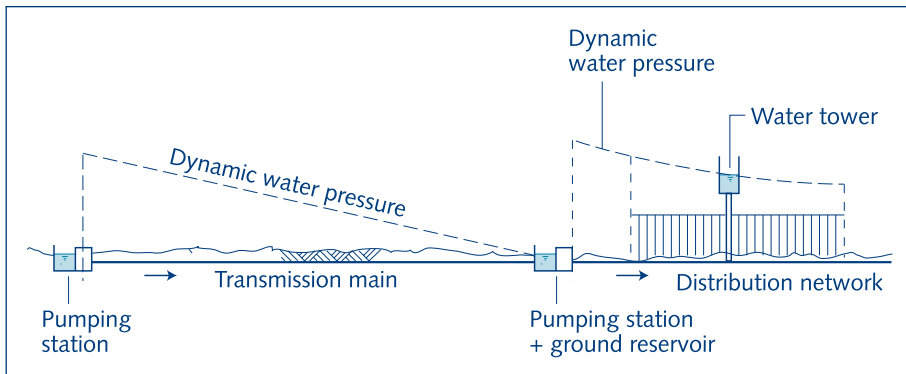


Fig. 21.13. Ground reservoir with pumping station combined with water tower

Sizeable ground reservoirs, of a few thousand cubic metres, are normally made of reinforced concrete. Smaller ones can be of mass concrete or brick masonry. Water towers are made of steel, reinforced concrete or brickwork on concrete columns. Steel tanks are mostly placed on a steel or wooden support framework.

Examples of small service reservoirs are shown in figures 21.14 and 21.15.

An elevated service reservoir (steel tank on brick masonry support) is shown in figure 21.16. Figure 21.17 features a reinforced brickwork tank supported by masonry walls.

21.4 Distribution system design

Design parameters

Design criteria for pressures, velocities and hydraulic gradients are similar to those for transmission pipes discussed in chapter 20, but set in a bit wider range because of demand variations.

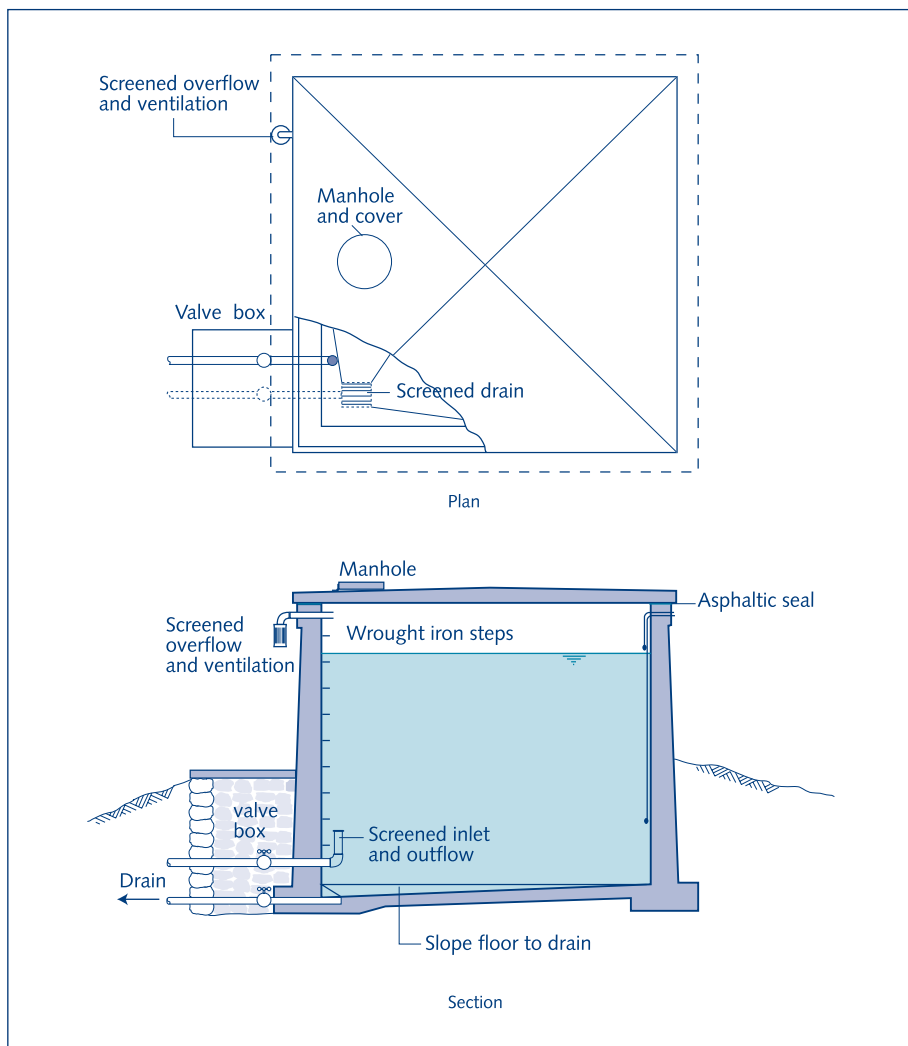


Fig. 21.14. Construction details of small reservoir

A typical range of velocities in distribution pipes is between 0.5 and 1.0 m/s, occasionally up to 2 m/s. The hydraulic gradients usually range between 1 and 5 m/km, occasionally up to 10 m/km. In case of smaller pipes, $D < 50$ mm, the hydraulic gradient can even be higher.

The pressure criterion is dependent on topographic conditions, availability of water at the source and overall condition of the pipes. The minimum pressures should not drop below 5-10 mwc. In larger distribution areas where water scarcity is not an issue, the minimum pressures can range between 20 and 30 mwc above street level, where there are house connections. This is sufficient for supply of 2-3 storey buildings.

Pressures higher than 60 mwc should be avoided in general, due to increased leakage and risks of bursts, especially in poorly maintained networks.

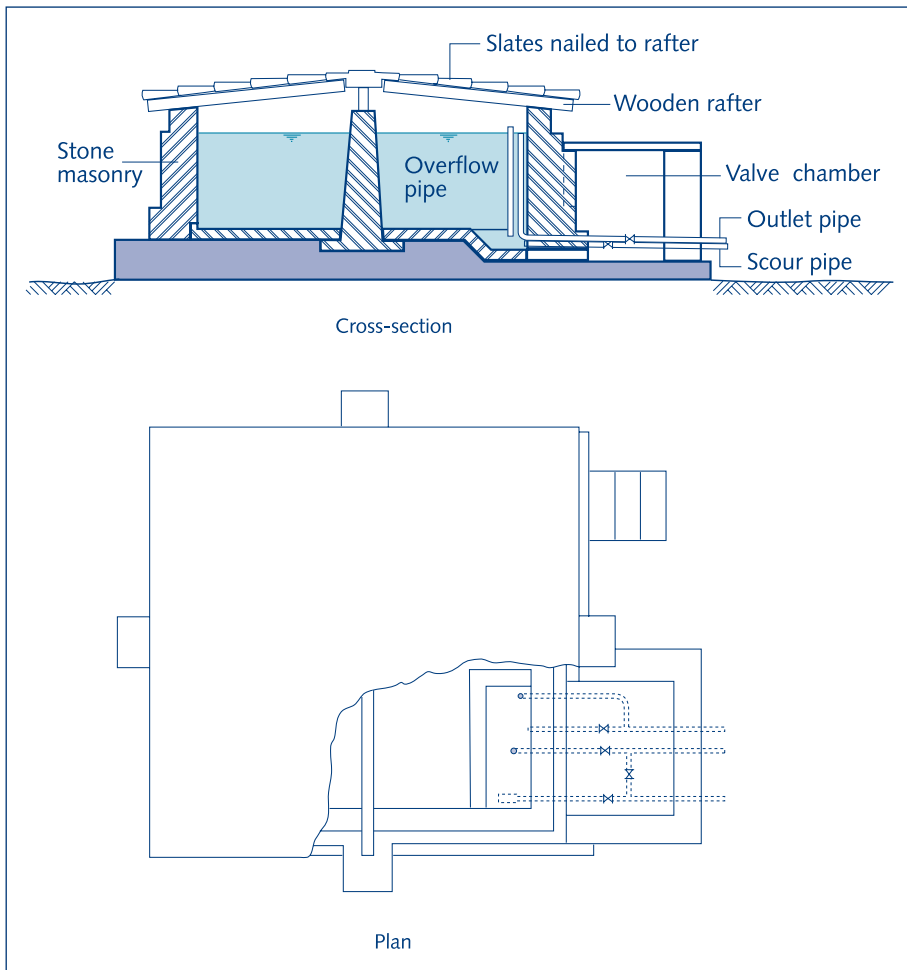


Fig. 21.15. Small service reservoir

Layout and nodal demands

After choosing a preliminary supply scheme (gravity/pumping) and type of network layout (branched/looped) with its main components, the distribution area is divided into a number of demand districts according to the topography, land use classification and density of population. Boundaries may be drawn along rivers, roads, high points or other features that distinguish each district. The secondary mains and distribution pipes can then be plotted on the plan.

Once all the sectors are fixed, the population for each district can be estimated or computed from data collected in the community involvement process. The water demand per district is then computed using per capita water usage figures for domestic water consumption and selected values for the other, non-domestic water requirements (including productive uses). This per capita usage figure may vary depending on the water service level that the community in the concerned district has decided on.

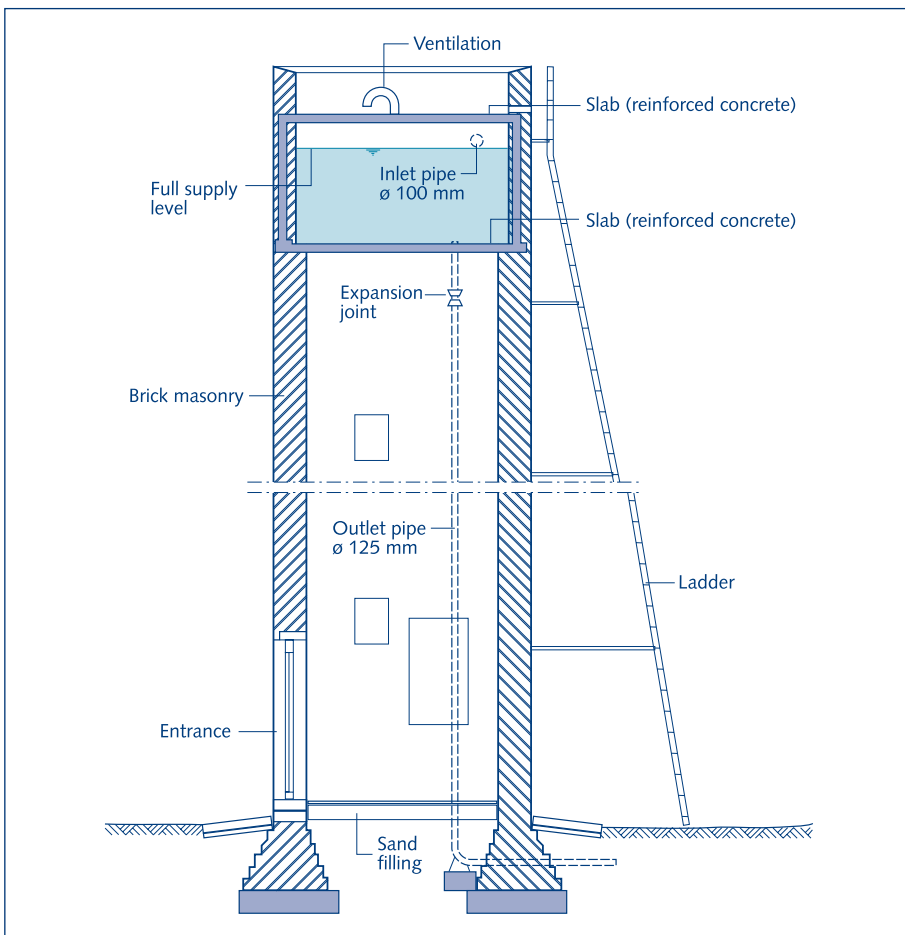


Fig. 21.16. Elevated service reservoir made of steel

The community process that can be followed, also to get realistic figures on water demand, is described in chapter 2. A certain leakage percentage is to be assumed too; some 10% of the water production is a fair assumption in the case of new systems.

Although in reality the water will be drawn off at many points along the length of the pipes, it is common engineering practice to assume that all demand is concentrated in the nodal points (i.e. pipe junctions) of the distribution network. The hydraulic calculation is much simplified by this assumption and the errors so introduced are acceptable.

For preliminary calculation of the nodal demands, a simple method may be employed using the water consumption rate per linear metre of distribution pipe. This rate is, of course, greatly influenced by the type of discharge points: public standpipes, yard taps,

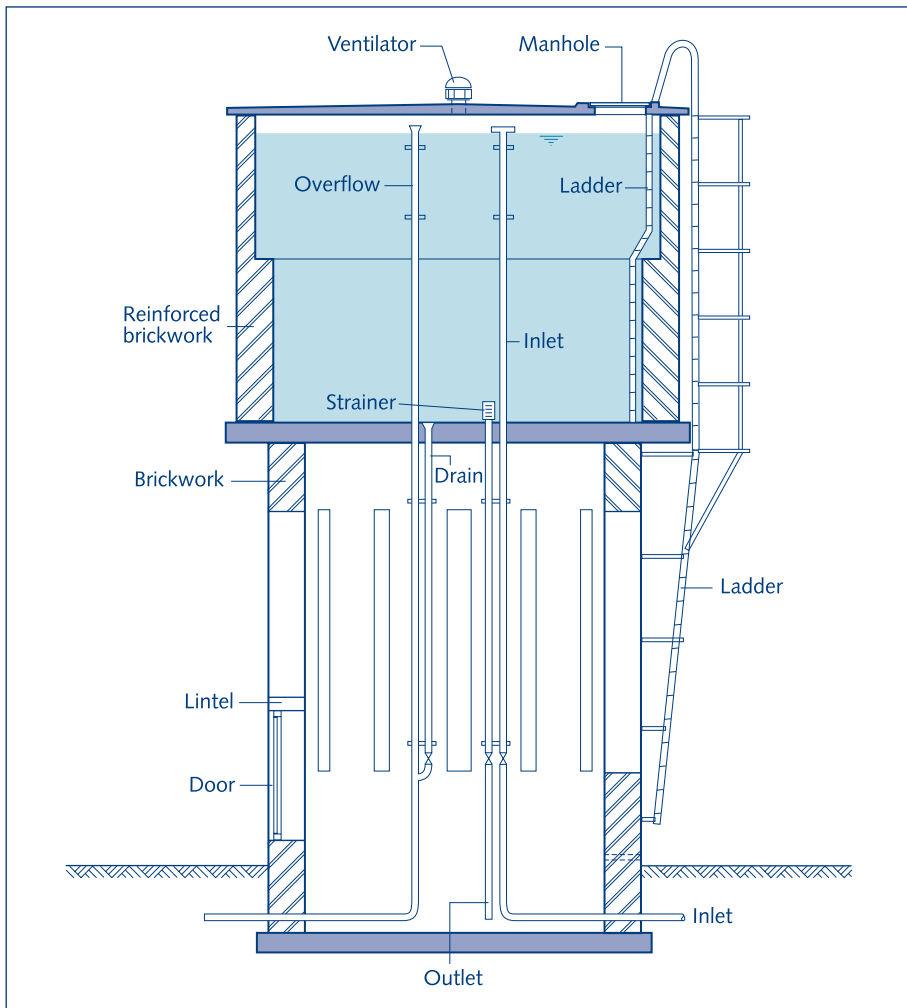


Fig. 21.17. Elevated service reservoir made of concrete

house connections, or combinations of these. Once the total demand alongside each pipe has been determined, it is going to be split between the two corresponding nodes. This approach, based on the assumption of even dispersion of the service connections, makes a good starting point for hydraulic calculations. How much it is really true is to be checked in the field. In addition, a separate demand node should be assigned to any major user in the system (school, hospital, etc.).

Having calculated demands at the nodal points, the required pipe diameters can be estimated. Based on the demand distribution, the main pipes should be laid towards the areas of higher demand, usually alongside the main traffic routes in the area.

Hydraulic calculation

If the network is of branched structure, the flows and their directions can easily be determined assuming the flow continuity in each node. For node n that is a junction of j pipes:

$$\sum_{i=1}^j Q_i - Q_n = 0$$

where Q_i is the pipe flow and Q_n the nodal demand. The equation is basically saying that the sum of all inflows and outflows in each node equals zero.

When the flow in each pipe is determined, the diameters will be adopted based on the hydraulic properties of the available manufactured sizes. The head loss calculation has to be done according to the principles explained in chapter 20. Lastly, the pressures in the system will be determined, either starting from the minimum required pressure in a specified (critical) node or from the known head at the supply point.

The following example illustrates this simplified design method (Fig. 21.18).

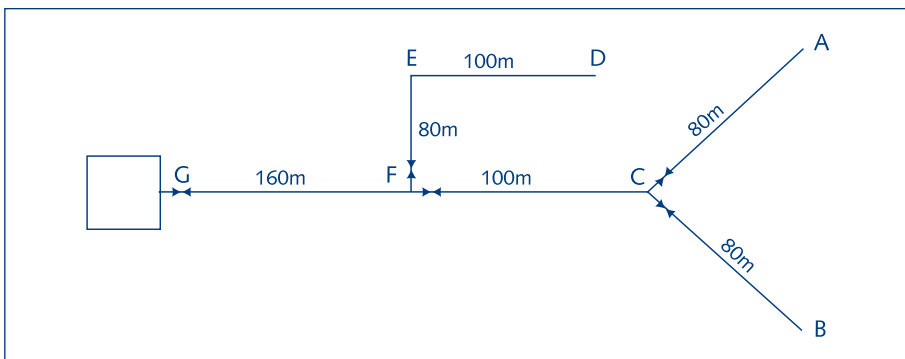


Fig. 21.18. Simple water distribution system (schematic)

Design data

Number of persons served:	1750
Total length of pipes:	600 m
Specific demand:	50 litres per person per day
Daily peak factor (k_1):	1.2
Hourly peak factor (k_2):	2.0

Calculation

Average flow carried by the distribution system: $Q_{avg} = 1750 \times 50 = 87500 \text{ l/d} = 1.0 \text{ l/s}$.

Peak flow carried by the system: $Q_{max} = 1.2 \times 2.0 \times 1.0 = 2.4 \text{ l/s}$.

Water use rate per length of distribution pipes: $q = 2.4/600 = 0.004 \text{ l/s/m}^1$.

Multiplying the length of each pipe by the unit flow rate gives the tentative demand along the pipe (table 21.1a).

Table 21.1a Tentative determination of pipe demands

Sections	Length L (m)	Piped demand $q \cdot L$ (l/s)
A-C	80	0.32
B-C	80	0.32
C-F	100	0.40
D-E	100	0.40
E-F	80	0.32
F-G	160	0.64
Total	600	2.40

Splitting the pipe demand between the corresponding nodes yields the nodal demands.

Table 21.1b Nodal demands

Node	A	B	C	D	E	F	G	Total
Q_n (l/s)	0.16	0.16	0.52	0.20	0.36	0.68	0.32	2.40

Consequently, the pipe flows will be determined from the continuity equation.

Table 21.1c Pipe flows

Pipe	A-C	B-C	C-F	D-E	E-F	F-G
Q (l/s)	0.16	0.16	0.84	0.20	0.56	2.08

In the next step the maximum carrying capacity can be computed for selected design velocity and range of available (manufactured) pipe diameters: $Q_{\max} = v \times D^2 \pi / 4$. In this example the maximum flow carried by the pipes for a design velocity of 0.75 m/s is shown in table 21.2. Comparing the pipe flows in table 21.1c with the maximum flows in table 21.2 leads to the selection of diameters. The hydraulic calculation proceeds with the adopted D-values. The results in table 21.3 are obtained by using the Darcy-Weisbach formula (see chapter 20), for $k = 0.1$ mm and $T = 20^\circ$. Finally, the pressures in the nodes will be calculated for known elevations (in metres above sea level), and assumed head at the supply point of 50 mwc (table 21.4).

Table 21.2 Pipe flow (for $v = 0.75$ m/s)

D (mm)	D (inch)	Q_{\max} (l/s)
30	1¼	0.53
40	1½	0.94
50	2	1.47
60	2½	2.12
80	3	3.77
100	4	5.89

Table 21.3 Head loss calculation ($k = 0.1$ mm, $T = 20^\circ$)

Sections	Pipe Flow Q (l/s)	Adopted D (mm)	Velocity v (m/s)	Hydr.grad. S (m/km)	Head loss SxL (mwc)
A-C	0.16	30	0.23	3.38	0.27
B-C	0.16	30	0.23	3.38	0.27
C-F	0.84	40	0.67	16.95	1.70
D-E	0.20	30	0.28	5.05	0.50
E-F	0.56	40	0.45	7.97	0.64
F-G	2.08	60	0.74	12.10	1.94

Calculation of looped systems is much more complicated, due to the fact that the flow rates and directions in the pipes are not initially known. Smaller looped networks can be disintegrated into branched type layouts by disconnecting selected peripheral pipes from some junctions (Fig. 21.19). This "imaginary disconnection" is done only for the sake of easier calculation and will result in smaller diameters for those pipes.

Table 21.4 Determination of pressures

Nodes	Elevation z (msl)	Head H (mwc)	Pressure p (mwc)
A	24.8	46.09	21.29
B	22.2	46.09	23.89
C	17.5	46.36	28.86
D	20.0	46.92	26.92
E	28.2	47.42	19.22
F	35.6	48.06	12.46
G (Source)	-	50.00	-

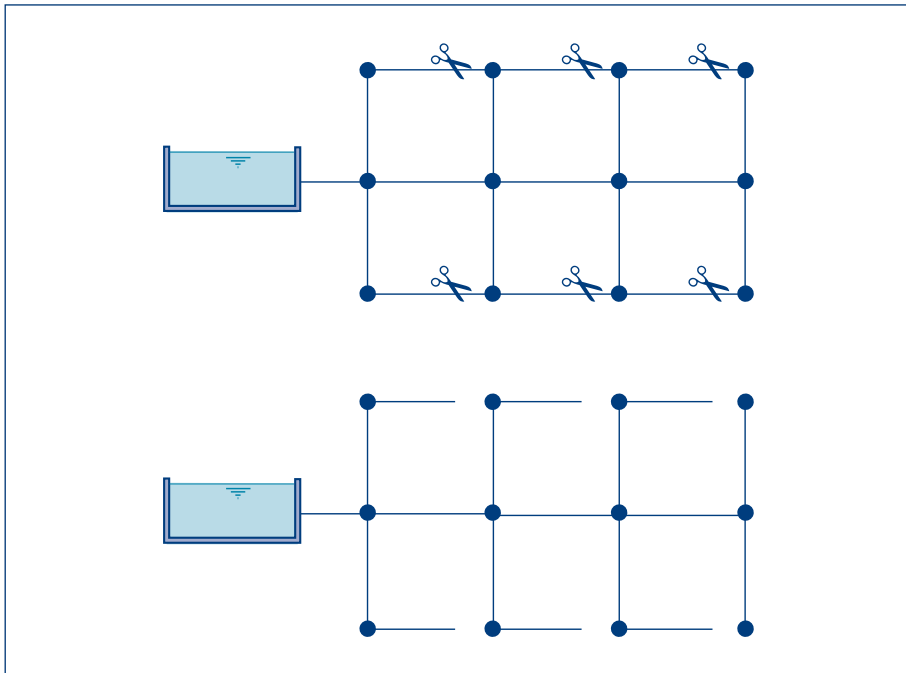


Fig. 21.19. Imaginary branching of looped layouts

Larger systems have to be considered as being looped. Apart from the continuity and head loss equation, an additional equation is required in this case:

$$\sum_{i=1}^j \Delta H_i = 0$$

The equation reflects the continuity of the hydraulic grade line in each loop of the network, i.e. the sum of the head losses being equal zero.

The methodology of the calculation of the looped networks was first developed by Hardy-Cross. In short, it consists of the following steps:

1. Estimated flows are initially assigned to each pipe, fitting the continuity equation in all nodes, as well as possible.
2. Based on these flows, the head loss is calculated in each pipe, as described in chapter 20.
3. The sum of the head losses along each loop is checked, respecting the orientation of flows (e.g. clockwise).
4. If the sum at any loop is out of requested accuracy $0 + \sum \Delta H$, the following flow correction is to be introduced for each pipe of that loop:

$$\delta Q_j = \frac{-\sum_{j=1}^n \Delta H_j}{2 * \sum_{j=1}^n \left| \frac{\Delta H_j}{Q_j} \right|}$$

5. The iteration procedure is then repeated with the new flows $Q + \delta Q$, until the accuracy is satisfied for all loops.
6. After the iteration of flows and head losses has been completed, the nodal pressures can be determined from the reference pressure.

Obviously the manual calculation is very time consuming even with only a few-loop system. Luckily the development of computer technology has allowed such iteration methods to be programmed, and the Hardy-Cross method was further developed as a basic algorithm for computer calculations. Since the 1990s, this has opened new possibilities for design of distribution networks through application of computer models.

Use of computer models

Widespread introduction of personal computers has enhanced hydraulic design of distribution networks. Commercial programmes available on the market, sometimes even free of charge, enable very precise and quick calculations, which makes them equally suitable for the design of simple rural systems or large urban networks of a few thousand pipes. Accessibility of such software and PCs to the engineers of developing countries has been significantly improved since the mid-nineties.

The computer programmes in use are all pretty similar in concept, with the following common features:

- PC-Windows™ based applications
- Allow extended period hydraulic simulations
- Possess integrated module for water quality simulations
- Handle virtually unlimited size of the network in any configuration
- Have excellent graphical interface for presentation of results

The main distinctions between these programmes are in specific formats of input data used, as well as in the way the calculation results are processed.

The modelling process consists of the following steps:

1. Input data collection
2. Network schematic
3. Model building
4. Model testing
5. Problem analysis

Input data collection

Powerful computational tools have enabled the focus to be shifted from the calculation to the collection of reliable input data. High quality information about demands, system dimensions and materials is crucial for accurate results. Operation and maintenance data are important too, in the case of rehabilitation of existing systems. This is often a limitation for the model. Well-conducted fieldwork data collection is therefore a very important initial step of the modelling procedure. The information to be investigated is listed below.

1. General

General layout of the network is taken from the maps available in the water distribution company. The specific data to look for are:

- 1.1 Topography - ground elevations in the area of the system; some specific natural barriers.
- 1.2 Type of the system - distribution scheme: gravity, pumping, combined; location and role of each system component.
- 1.3 Population - distribution and estimated growth.

2. Water demand

- 2.1 Demand categories present in the system: domestic, industry, tourism, etc.
- 2.2 Average consumption, patterns of variation: daily, weekly and seasonal.
- 2.3 Type of domestic water use: direct supply, roof tanks, etc.; average household size; habits with respect to the water use.
- 2.4 Demand forecasting.

3. Network layout

- 3.1 Nodes (discharge points) - concerns predominantly the supply points of at least a few hundred consumers or major industry. Relevant for each point are
 - location (X,Y) in the system
 - ground elevation (Z)
 - average consumption and dominant category(ies)

- 3.2 Pipes - concerns predominantly the pipes, $D > 50$ mm. Relevant for each pipe are
 - length
 - diameter (internal)
 - material and age
 - assessment of corrosion level (k or C value, if available)
- 3.3 Service reservoirs - type (ground, elevated), capacity, minimum and maximum water level, shape (e.g. through the "volume-depth" curve).
- 3.3a Individual roof tanks (where applicable) - type and height of the tank, capacity, inflow/outflow arrangement, average number of users per house connection, description of house installations (existence of direct supply in the ground floor).
- 3.4 Pumping stations - number and type of pumps; duty head and flow and preferably the pump characteristics for each unit; age and condition of pumps.
- 3.5 Others - description of appurtenances that may significantly influence the system operation (e.g. valves, measuring equipment, etc.).

4. System operation & monitoring

Important (and preferably simultaneous) measurements for calibration of the model are:

- pressure in a few points covering the entire network
- level variations in the service reservoirs and roof tanks (where applicable)
- pressures and flows in the pumping stations
- flows in a few main pipes in the network
- valve operation (where applicable)

All this information may not be easy to collect. However, some knowledge about the system should exist, even in descriptive form. For instance, in which period of the day is a certain reservoir empty (full), a certain pump on (off), a certain valve open (closed), a certain consumer with (without) water or with (without) sufficient pressure, etc. Where there is a possibility of continuous measurements, typical days should be compared: the same day of the week in various seasons, or various days of the week in the same season.

5. System maintenance

Type of maintenance, water metering, the unaccounted-for water level and sources (leakage, faulty water meters, illegal connections, etc.), water quality in distribution network.

6. Water company

Organisation, facilities, practice, plans for future extension of the system. This information can be used to opt for a certain degree of reliability i.e. reserve capacity in the system. It may also have implications on the way the phased development should be approached. Some components of the system can deliberately be oversized to cater for future development of particular areas or connection to a new water source.

Network schematic

Hydraulic calculation of looped networks is based on systems of equations with a complexity directly proportional to the size of the system. Thus, some schematisation (also called skeletonisation) is necessary up to the level where the model accuracy will not be substantially affected, enabling quicker calculations at the same time.

The answers should be reached with the minimum necessary number of pipes and nodes. Expanding the model by the system parts that have no significant impact on its operation only adds to the calculation time and the volume of printout.

Model building

Just as in reality, it is advisable to build the network model in stages, increasing the level of detail gradually. Starting immediately with the full-size network with all components included will almost certainly yield lots of troubles during the testing of the model.

In many cases the input file format has to be obeyed strictly; this is the only code the programme can understand while reading the data. Making errors during the model building is therefore common. Some programmes allow input in an interactive way so the chance of false network configuration will be diminished. In other cases the error check and warning messages will be activated after running the calculation.

Model testing

Once the first simulation run is completed the immediate concern is whether the result matches the reality. In this phase several runs have to be executed that should confirm that

- the model gives a logical response to any altering of the input data (model validation)
- the model's behaviour corresponds to the reality (model calibration)

The reasons for any inaccuracy may be that

- some input data were badly estimated because of lack of the field data
- the network is wrongly interpreted by the model due to possible typing errors, omitted data or inaccurate field measurements
- format of the input file is incorrect but the error is 'unknown' to the error library of the programme

It should never be forgotten that the computer models cannot match the real situation totally; the results should be judged based on the quality of the input data and the measurements used for model calibration.

Problem analysis

With correct execution of all other steps, the real analysis of the problem is the final and the shortest step. After a few simulations a well-managed model will show the full picture of the selected alternative.

One of the most popular computer programmes in the market at present is EPANET made by the US Environmental Protection Agency. The programme possesses all of the above listed features and can be downloaded from the Internet, free of charge. The site can be accessed through any search engine by typing keyword 'epanet'.

Graphical layout of the simulation results in EPANET is shown in figure 21.20.

21.5 Operation and maintenance of distribution systems

Unaccounted-for water

The amount of water that can be billed will always be smaller than the amount supplied. Moreover, the water actually passing through the taps is also smaller than the amount supplied, be it charged or not. The difference in the first case refers to the unaccounted-for water (UFW) while the second one represents leakage. Thus, the leakage is a component of UFW. Other important sources can be faulty water meters, illegal connections, poor education of consumers, etc.

UFW is an important element of water demand and a great concern of many water companies. In some systems UFW is the most significant "consumer", reaching up to 50% of the total water supply. There can be various ways of fighting this problem but due to high costs of such programmes the real consideration tends to start only when the UFW levels exceed 20-30%. Limited capacity of the source can also be an important factor in such cases. Water conservation is increasing in importance as more and more regions begin to experience serious water shortages, and reducing UFW is a good way to start.

Leakage is usually the most significant contributor to high UFW levels. The factors influencing leakage are

- soil characteristics, soil movement, traffic loading
- defects in pipes, poor quality of joints
- poor quality of workmanship, damage due to excavation for other purposes
- pipe age and corrosion level
- high pressures in the system
- extreme temperatures

There are a variety of managerial and technical ways to address the UFW problem. Managerial measures include the following:

- Regular checks by caretakers or by alerts from consumers for pipeline damage, leakage and illegal connections
- Regular checks by the water point committee, caretakers and technicians on the quality and leakage of connections (also the presence of illegal connections), meters (if present) and taps

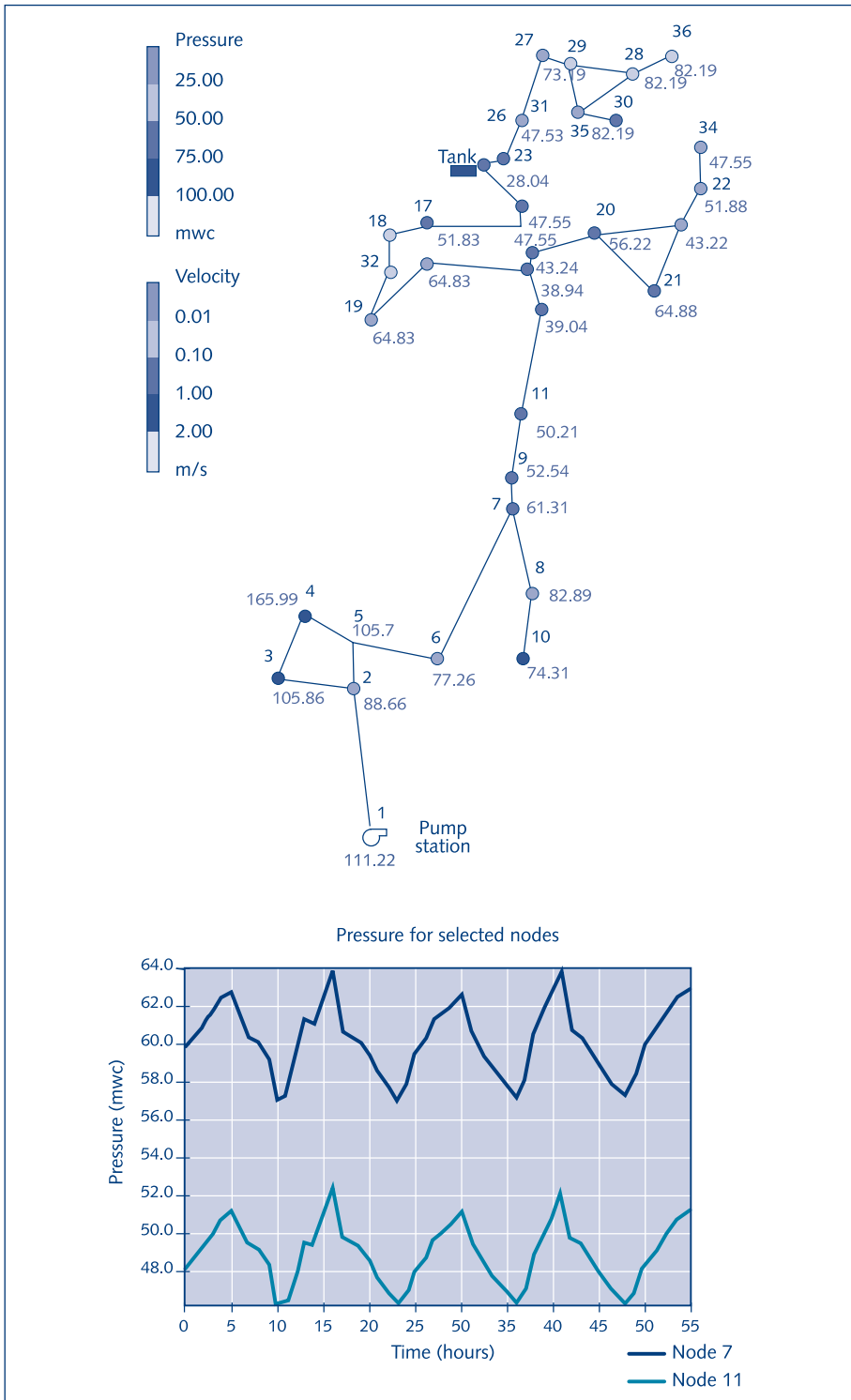


Fig. 21.20. Layout of the EPANET calculation results

- Arrangements at the water-vending points, public taps and group connections to minimise the waste of water by good caretaking

Global estimates of leakage levels come from an annual balance of the delivery and metered consumption for the whole network. Bursts of main pipes can be detected by the flow measurements at supply points (Fig.21.21).

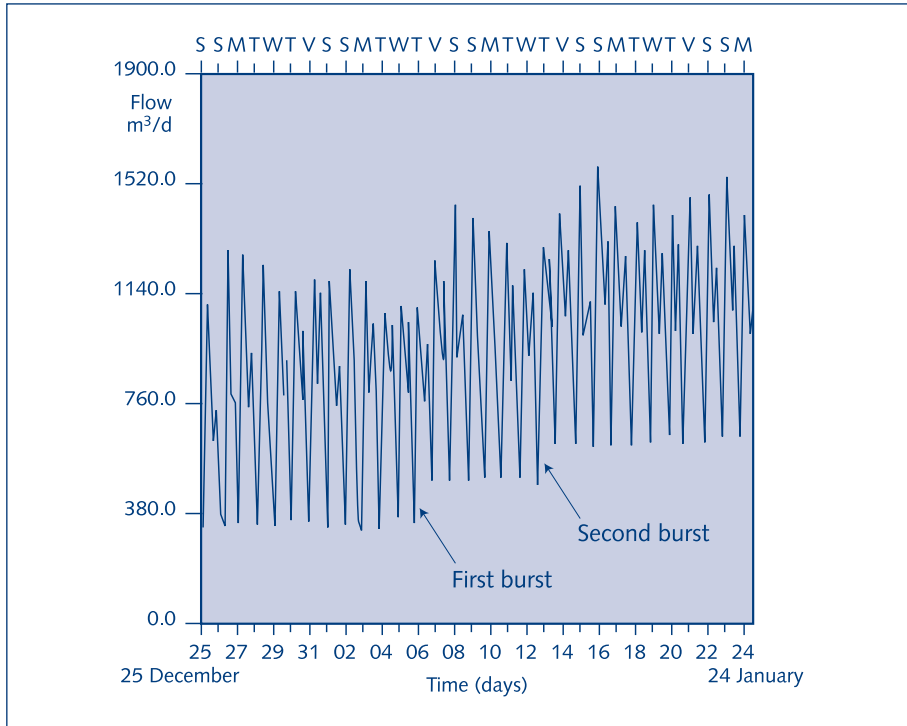


Fig. 21.21. Leak detection from the flow measurements

For more detailed analysis of the leakage, suspected parts of the system have to be inspected during several hours or days, depending on the size of controlled area. Those temporary measurements are usually carried out overnight, when real consumption and overall noise level are minimal. The area is isolated from the rest of the system by closing the border valves and its inflow and outflow are measured. Some knowledge about the actual night consumption should exist in advance; flows detected on top of that are part of the UFW, mostly leakage.

Measurements are repeated at weekly intervals for a period of a few months. Any pipe burst between two measurements would be reflected in sudden increase in registered demand.

Average leakage level can also be estimated by monitoring pressures in the system. A sudden drop of pressure could also indicate a major pipe failure. Under normal conditions night pressures should be kept as low as possible in order to reduce the leakage levels.

Finding a precise leak location can be a difficult problem. In case of severe breaks the water may appear on the surface and the exact position of the leak can be determined by drilling test holes alongside the pipe route. If the leak is not visible on the surface, leak detection equipment has to be used. The most common devices are an acoustic (sound) detector and a leak noise correlator.

Acoustic detectors rely on sounding directly on the pipe or fitting, or indirectly on the ground surface. The noise generated from the leak is transmitted by the receiver attached to a stick, to the amplifier connected to a stethoscope (Fig. 21.22). This method is not always reliable; leaks at lower pressures and specifically those in plastic pipes may produce undetectable noise. However, with skilled personnel working under silent (night) conditions, some 70-80% of the leaks should be discovered.

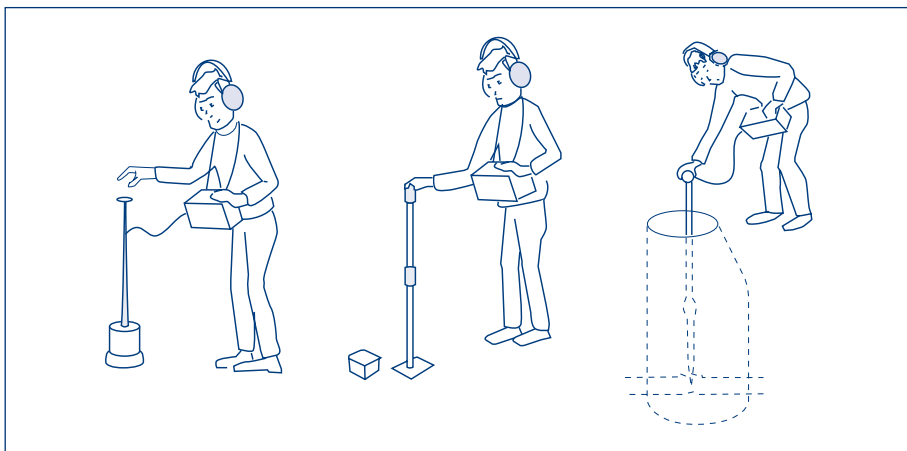


Fig. 21.22. Leak location by using sound detector

Leak noise correlators detect the exact burst location by registering the noise spreading through the water. By placing microphones at the ends of the controlled pipe section (up to a few hundred metres), the difference in time required for the leak noise to reach the microphones can be measured. The leak position can then be calculated from the known length of the section. This method is very effective in detecting leaks under background noise levels. However, it may be less accurate when being used in sections with plastic pipes or with more than one leak.

Faulty water meters are the second main source of unaccounted-for water. Typical water meters register flows with average accuracy of about 2%, when they are new. However, this error becomes higher for small flows, below 50 l/h. When not properly maintained the water meter may register flows with errors between 20 and 40% after a couple of years in service. This lack of accuracy can cause serious revenue losses.

Complicated methods of monitoring and leak detection would normally not be employed in small community water supply schemes. They require expensive equipment and trained personnel. Even so, installing at least a few measuring devices at the right points in the network may be of great help in collecting information about the system operation. The minimum is to have flow and/or pressure meters in the pumping stations. Water levels in the reservoirs should also be observed at regular intervals during the day. Ideally, a few pressure gauges should be installed within the network.

Corrosion

Corrosion of metal pipes is one of the major causes of poor operation of water distribution systems. It appears as a result of reaction between the water and metal. This internal corrosion causes three problems:

- The pipe mass is lost through oxidation to soluble iron, resulting in increased rate of pipe bursts
- The second by-product of oxidation is iron-bearing scale that accumulates on the pipe wall in the form of tubercles, causing a reduction of pipe capacity (head loss increase)
- Both soluble and particulate iron affects the water quality creating colour problems ('brown' or 'red' water)

External corrosion is a result of aggressive soils and may also contribute to the pipe burst rate to a large extent.

To reduce corrosion levels, metal pipes need to have internal and external coatings. Ductile iron and steel pipes are normally delivered with internal cement lining and external coatings made of plastic, epoxy, or bitumen. Steel pipes in pumping stations will usually be protected by painting. Handling of pipes during transport and laying has to avoid damage to the coatings. Once in service the cement lining can be dissolved due to leaching of calcium at low pH values. High turbulence or sudden change of the flow direction at high velocities may also damage the coating.

Water quality adjustment is the easiest and most practical way to make water non-corrosive. However, it is not always effective because of possible differences in water quality at the sources. Two basic methods are pH correction and oxygen reduction. Chemicals commonly used for pH adjustment are lime, caustic soda or sodium

(bi)carbonate. Oxygen removal is rather expensive but some control measures can be introduced through optimisation of aeration processes and sizing of well and distribution pumps that will avoid air entry.

Other options, such as adding inhibitors or cathodic protection of the pipes are too complex and expensive for small distribution schemes.

Bad design of the pipes and structures may cause severe corrosion even in materials that are highly resistant. Some of the important design considerations include

- selection of appropriate flow velocity
- selection of appropriate metal thickness
- reduction of mechanical stresses
- avoiding sharp bends and elbows
- avoiding grounding of electrical circuits to the system
- providing easy access to the structure for periodic inspection, maintenance and replacement of damaged parts

Pipe cleaning and disinfection

Corrosion deposits in pipes or sediments caused by improper treatment have to be removed to prevent water quality deterioration. Three techniques commonly used are flushing, swabbing and air scouring. These techniques may greatly help to improve the water quality but additional maintenance equipment has to be employed. Moreover, the network layout needs to include a number of hydrants or washouts to connect this equipment.

Flushing is the simplest method of cleaning but with some disadvantages:

- Large amounts of water used (particularly in large diameters)
- It is less effective in low pressure areas
- It may disturb flow patterns upstream of the cleaned section
- In areas with active corrosion, it offers only a temporary improvement

The efficiency of flushing can be increased by injection of compressed air into a continuous but smaller flow of water. Pushed by the air, the water will form into discrete slugs forced along the pipe at high velocities. The length that can be effectively cleaned by air scouring depends on:

- available static pressure (higher pressure - longer length)
- pipe friction (higher friction - shorter length)
- compressor size (larger compressor - longer length)
- pipe gradient (longer lengths when cleaning uphill)

Swabbing is a technique where a cylindrical swab is inserted into the pipe and driven along by the water pressure pushing the deposits ahead. The swab is porous and allows about 10% of the water flow to pass, which aids the transport of the deposits. Swabs are usually made of polyurethane of different hardness and construction for various degrees of reduction in pipe cross-section. Recommended travelling speed of the swab is 0.3-1.2 m/s.

The problem of animal populations appearing in water distribution systems is predominantly aesthetic and it is therefore a matter of maintaining it at such a level that the consumer is unaware of their presence. Pipe disinfection can be done either by cleaning or by chemical treatment. Swimming animals can be removed relatively easily by flushing. Chemical treatment is carried out where the flushing is insufficient. The chemicals commonly used are chlorine, pyrethrins and permethrin. Pyrethrins and permethrin are toxic to fish, so they should be used and disposed of very carefully.

When using chlorine, higher concentrations are required than the normal dosages in water leaving the treatment plant. The concentrations applied during pre-chlorination may be effective in reducing animal appearance in the treatment works. An infestation in the distribution system can be controlled in most cases by maintaining 0.5-1.0 mg/l of residual chlorine for a week or two.

Long-term measures include removal of organic matter (restricting nutrients for the animals), which can be achieved by the following methods:

- Improvement of the treatment process regarding suspended solids removal and animal penetration
- Periodic cleaning of pipes and service reservoirs
- Maintenance of a chlorine residual throughout the distribution system
- Proper protection of openings on service reservoirs
- Elimination of dead ends and stagnant waters where ever possible.

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