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Introduction to Urban Water Distribution

Nemanja Trifunović

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The more we learn, the less we know as we realise how much is yet to be discovered.

Contents

Preface

This book comprises the training material used in the three-week module 'Water Transport and Distribution 1', which is a part of the 18-month Master of Science programme in Water Supply Engineering Specialisation at UNESCO-IHE Institute for Water Education in Delft, The Netherlands. Participants in the programme are professionals of various backgrounds and experience, mostly civil or chemical engineers, working in water and sanitation sector from over 40, predominantly developing, countries from all parts of the world. To make a syllabus that would be relevant to such a heterogeneous group and ultimately equip them with knowledge to be able to solve their practical problems was quite a challenge.

The development of the materials started in 1994 based on the existing lecture notes made by J. van der Zwan (KIWA Institute) and M. Blokland (IHE) in 1989. Their scope was widened by incorporating the ideas and materials of K. Hoogsteen (Drenthe Water Company) and T. van den Hoven (KIWA Institute), prominent Dutch water distribution experts and then the guest lecturers of IHE.

The text was thoroughly revised in 1998 and further expanded by adding the workshop problems. In 2000, the design exercise tutorial was prepared, and finally in 2003 a set of so-called spreadsheet hydraulic lessons was developed for better understanding of the basic hydraulic concepts, and as an aid to solving the workshop problems. All these improvements were geared not only by developments in the subject, but also resulted from a search for the optimal method in which the contents could be understood within a couple of weeks. The way the lecture notes grew was derived from lively discussions that took place in the classroom. The participants reacted positively to each new version of the materials, which encouraged me to integrate them into a book for a wider audience.

During the work on the book, I came into contact with a number of UNESCO-IHE guest lecturers who also helped me with useful material and suggestions. J. Vreeburg (KIWA Institute & Delft University of Technology) and J. van der Zwan reviewed the draft text, whilst many interesting discussions were carried out with several other Dutch water supply experts, most recently with C. van der Drift (Municipal Water

Company of Amsterdam) and E. Arpadzić (Water company 'Evides' in Rotterdam). Giving lectures in Delft and abroad on various occasions, where similar programmes were also taking place, has allowed me to learn a lot from interaction with the participants who brought to my attention many applications and practices that differ from European practice.

Last but not least, I wish to mention D. Obradović from Belgrade University, a long-serving guest lecturer at UNESCO-IHE, whose materials were also used in this book. Prof Obradović was a pioneer of water distribution network modelling in former Yugoslavia, an advisor of Wessex Water PLC in UK, and the author of numerous publications and books on this subject. Sadly, he passed away just a few days before the first draft of the text was completed.

Nemanja Trifunović

Introduction

The book was written with the idea of elaborating general principles and practices in water transport and distribution in a practical and straightforward way. The most appropriate readers are expected to be amongst those who know little or nothing about the subject. Experts dealing with advanced problems can use it as a refresher of their knowledge.

The general focus in the contents is on understanding the hydraulics of distribution networks, which has become increasingly relevant after the massive introduction of computers and the exponential growth of computer model applications, also in developing countries. This core is handled in Chapter 3 that discusses the basic hydraulics of pressurised flows, and Chapter 4 that talks about principles of hydraulic design and computer modelling applied in water transport and distribution. Exercises resulting from these chapters are given in Appendices 1 (workshop problems), and 2 (design tutorial), respectively.

The main purpose of the exercises is to develop a temporal and spatial perception of the main hydraulic parameters in the system for given layout and demand scenarios. The workshop problems are a collection of simple problems discussing various supply schemes and network configurations in a vertical cross-section. Manual calculation is recommended here, whilst the spreadsheet lessons illustrated in Appendix 5 help in checking the results and generating new problems. On the other hand, the tutorial in Appendix 2 discusses, step by step, a computer-aided network design looking at the network in a plan i.e. from a horizontal perspective. To facilitate the calculation process, the EPANET software of the US Environmental Protection Agency has been used as a tool. This programme is becoming more and more popular worldwide, owing to its excellent features, simplicity and free distribution via the Internet.

Furthermore, the book contains a rather detailed discussion on water demand (Chapter 2), which is a fundamental element of any network analysis, and chapters on network construction (Chapter 5) and operation and maintenance (Chapter 6).

Complementary to these contents, more on the maintenance programmes and management issues in water distribution is taught by the Water Services Management scientific core group at UNESCO-IHE. Furthermore, the separate subjects on pumping stations, geographical information systems, water quality and transient flows, all with appropriate lecture notes, make an integral part of the 6-week programme on water transport and distribution, which explains the absence of these topics from the present version.

The book comes with a CD containing the spreadsheet hydraulic lessons, a copy of the EPANET software (Version 2.10) and the entire batch of the input files mentioned in the tutorial of the exercise in Appendix 2. Hence, studying with a PC will certainly help to master the contents faster. All applications are made to run on a wide range of PCs and MS Windows operating systems.

The author and UNESCO-IHE are not responsible and assume no liability whatsoever for any results or any use made of the results obtained based on the contents of the book, including the CD. On the other hand, any notification of possible errors or suggestion for improvement will be highly appreciated.

CHAPTER 1

Water Transport and Distribution Systems

1.1 INTRODUCTION

Everybody understands the importance of water in our lives; clean water has already been a matter of human concern for thousands of years. It is a known fact that all major early civilisations regarded an organised water supply as an essential requisite of any sizeable urban settlement. Amongst the oldest, archaeological evidence on the island of Crete in Greece proves the existence of water transport systems as early as 3500 years ago, while the example of pipes in Anatolia in Turkey points to water supply systems approximately 3000 years old (Mays, 2000).

The remains of probably the most remarkable and well-documented ancient water supply system exist in Rome, Italy. Sextus Julius Frontinus, the water commissioner of ancient Rome in around the first century AD, describes in his documents nine aqueducts with a total length of over 420 km, which conveyed water for distances of up to 90 km to a distribution network of lead pipes ranging in size from 20 to 600 mm. These aqueducts were conveying nearly 1 million m^3 of water each day, which despite large losses along their routes would have allowed the 1.2 million inhabitants of ancient Rome to enjoy as much as an estimated 500 litres per person per day (Trevor Hodge, 1992).

Nearly 2000 years later, one would expect that the situation would have improved, bearing in mind the developments of science and technology since the collapse of the Roman Empire. Nevertheless, there are still many regions in the world living under water supply conditions that the ancient Romans would have considered as extremely primitive. The records on water supply coverage around the world at the turn of the twentieth century are shown in Figure 1.1. At first glance, the data presented in the diagram give the impression that the situation is not alarming. However, the next figure (Figure 1.2) on the development of water supply coverage in Asia and Africa alone, in the period 1990–2000, shows clear stagnation. This gives the impression that these two continents may be a few generations away from reaching the standards of water supply in North America and Europe. Expressed in numbers, there are approximately one billion people in the world who are still living without access to safe drinking water.

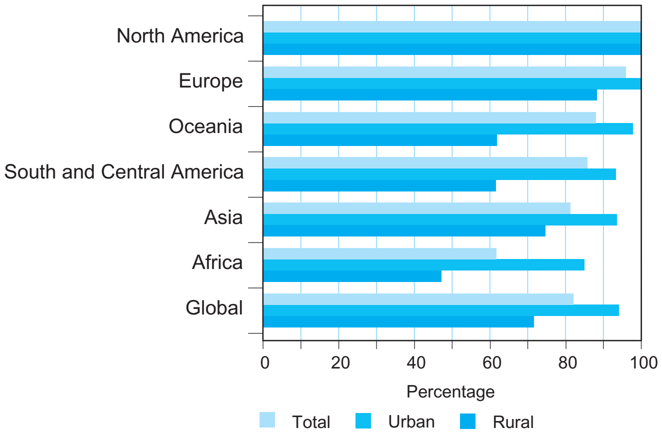


Figure 1.1. Water supply coverage in the world (WHO/UNICEF/WSSCC, 2000).

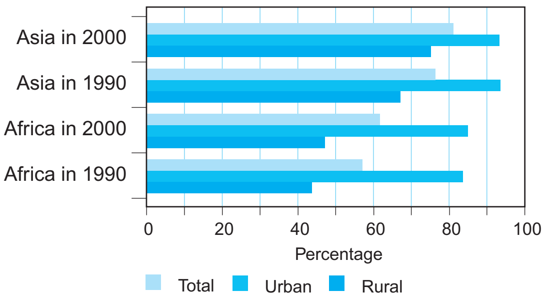


Figure 1.2. Growth of water supply coverage in Africa and Asia between 1990–2000 (WHO/UNICEF/WSSCC, 2000).

The following are some examples of different water supply standards worldwide:

1 According to a study done in The Netherlands in the late eighties (Baggelaar *et al.*, 1988), the average frequency of interruptions affecting the consumers is remarkably low; the chance that no water will run after turning on the tap is once in 14 years! Despite such a high level of reliability, plentiful supply and affordable tariffs, the average domestic water consumption in The Netherlands rarely exceeds 130 litres per person per day (VEWIN, 2001).

2 The frequency of interruptions in the water supply system of Sana'a, the capital of the Republic of Yemen, is once in every two days. The consumers there are well aware that their taps may go dry if kept on longer than necessary. Due to the chronic shortage in supply, the water has to be collected by individual tanks stored on the roofs of houses. Nevertheless, the inhabitants of Sana'a can afford on average around 90 litres each day (Haidera, 1995).

3 Interruption of water supply in 111 villages in the Darcy district of the Andhra Pradesh State in India occurs several times a day. House

connections do not exist and the water is collected from a central tank that supplies the entire village. Nevertheless, the villagers of the Darcy district are able to fetch and manage their water needs of some 50 litres per person per day (Chiranjivi, 1990).

All three examples, registered in different moments, reflect three different realities: urban in continental Europe with direct supply, urban in arid area of the Middle East with intermittent supply but more or less continuous water use, and rural in Asia where the water often has to be collected from a distance. Clearly, the differences in the type of supply, water availability at source and overall level of infrastructure all have significant implications for the quantities of water used. Finally, the story has its end somewhere in Africa, where there is little concern about the frequency of water supply interruptions; the water is fetched in buckets and average quantities are a few litres per head per day, which can be better described as 'a few litres *on* head per day', as Figure 1.3 shows.

The relevance of a reliable water supply system is obvious. The common belief that the treatment of water is the most expensive process in those systems is disproved by many examples. In the case of The Netherlands, the total value of assets of water supply works, assessed in 1988 at a level of approximately US$5 billion, shows a proportion where more than a half of the total cost can be attributed to water transport and distribution facilities including service connections, and less than half is apportioned to the raw water extraction and treatment (Figure 1.4). More

Figure 1.3. Year 2000 somewhere in Africa.

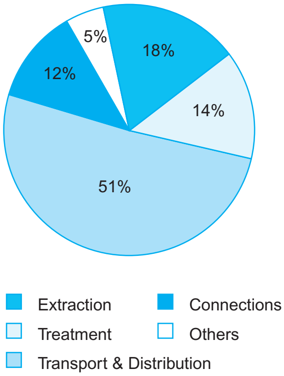


Figure 1.4. Structure of assets of the Dutch water supply works (VEWIN, 1990).

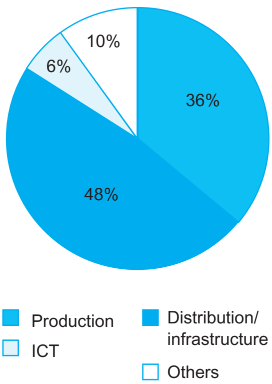


Figure 1.5. Annual investments in the Dutch water supply works (VEWIN, 2001).

recent data on annual investments in the reconstruction and expansion of these systems, presently at a level of approximately US$0.5 billion, are shown in Figure 1.5.

The two charts for The Netherlands are not unique and are likely to be found in many other countries, pointing to the conclusion that transport and distribution are dominant processes in any water supply system. Moreover, the data shown include capital investments, without exploitation costs, which are the costs that can be greatly affected by inadequate design, operation and maintenance of the system, resulting in excessive water and energy losses or deterioration of water quality on its way to consumers. Regarding the first problem, there are numerous examples of water distribution systems in the world where nearly half of the total production remains unaccounted for, and where a vast quantity of it is physically lost from the system.

Dhaka is the capital of Bangladesh with a population of some 7 million, with 80% of the population being supplied by the local water company and the average daily consumption is approximately 117 litres per person (McIntosh, 2003). Nevertheless, less than 5% of the consumers receive a 24-hour supply, the rest being affected by frequent operational problems. Moreover, water losses are estimated at 40% of the total production. A simple calculation shows that under normal conditions, with water losses, say at a reasonable level of 10%, the same production capacity would be sufficient to supply the entire population of the city with a unit quantity of approximately 140 litres per day, which is above the average in The Netherlands.

Hence, transport and distribution systems are very expensive even when perfectly designed and managed. Optimisation of design, operation and maintenance has always been, and will remain, the key challenge of any water supply company. Nowadays, this fact is underlined by the population explosion that is expected to continue in urban areas, particularly of the developing and newly industrialised countries in the coming years. According to the survey shown in Table 1.1, nearly five billion people will be living in urban areas of the world by the year 2030, which

Table 1.1. World population growth 1950–2030 (UN, 2001).

Region	Total population (millions)/urban population (%)			
	1950	1975	2000	2030
North America	172/64	243/74	310/71	372/84
Europe	547/52	676/67	729/75	691/83
Oceania	13/62	21/72	30/70	41/74
South and Central America	167/41	322/61	519/75	726/83
Asia	1402/17	2406/25	3683/37	4877/53
Africa	221/15	406/25	784/38	1406/55
Global	2522/28	4074/38	6055/47	8113/60

Table 1.2. World urban population growth 1975–2015 (UN, 2001).

Areas	Population (millions)/% of total		
	1975	2000	2015
Urban, above 5 million inhabitants	195/5	418/7	623/9
Urban, 1 to 5 million inhabitants	327/8	704/12	1006/14
Urban, below 1 million inhabitants	1022/25	1723/28	2189/31
Rural	2530/62	3210/53	3337/46
Total	4074/100	6055/100	7154/100

is over 70% more than in the year 2000 and three times as many as in 1975. The most rapid growth is expected on the two most populated and poorest continents, Asia and Africa, and in large cities with between one and five million and those above five million inhabitants, so-called mega-cities, as Table 1.2 shows.

It is not difficult to anticipate the stress on infrastructure that those cities are going to face, with a supply of safe drinking water being one of the major concerns. The goal of an uninterrupted supply has already been achieved in the developed world where the focus has shifted towards environmental issues. In many less developed countries, this is still a dream.

1.2 DEFINITIONS AND OBJECTIVES

1.2.1 *Transport and distribution*

In general, a water supply system comprises the following processes (Figure 1.6):

1 raw water extraction and transport,
2 water treatment and storage,
3 clear water transport and distribution.

Transport and distribution are technically the same processes in which the water is conveyed through a network of pipes, stored intermittently and pumped where necessary, in order to meet the demands and pressures in the system; the difference between the two is in their objectives, which influence the choice of system configuration.

Water transport systems

Water transport systems comprise main transmission lines of high and fairly constant capacities. Except for drinking water, these systems may be constructed for the conveyance of raw or partly treated water. As a part of the drinking water system, the transport lines do not directly serve consumers. They usually connect the clear water reservoir of a treatment

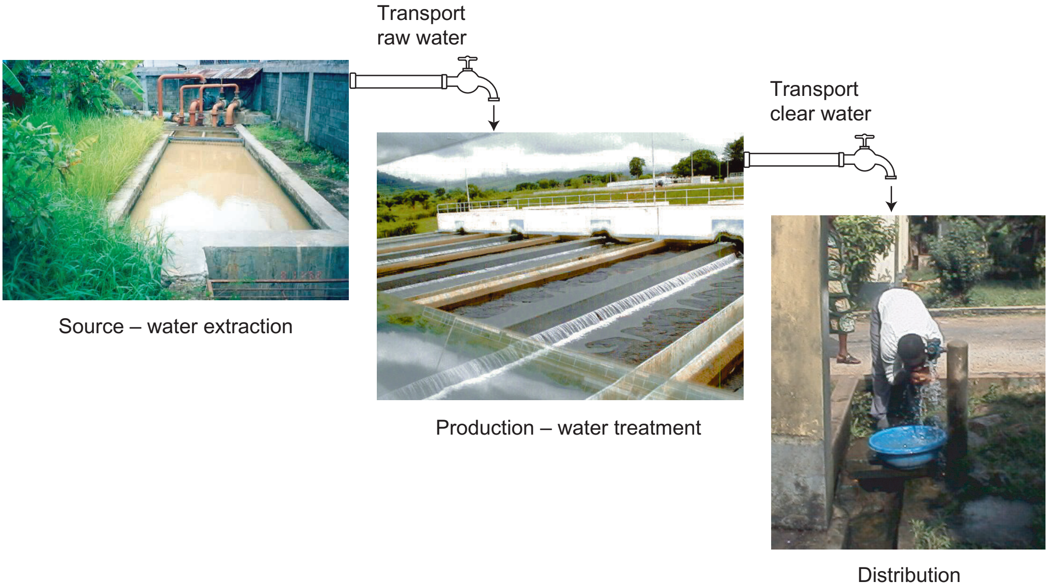


Figure 1.6. Water supply system processes.

plant with some central storage in the distribution area. Interim storage or booster pumping stations may be required in the case of long distances, specific topography or branches in the system.

Branched water transport systems provide water for more than one distribution area forming a regional water supply system. Probably the most remarkable examples of such systems exist in South Korea. The largest of 16 regional systems supplies 15 million inhabitants of the capital Seoul and its satellite cities. The 358 km long system of concrete pipes and tunnels in diameters ranging between 2.8 and 4.3 metres had an average capacity of 7.6 million cubic metres per day (m^3/d) in 2003.

However, the largest in the world is the famous 'Great Man-made River' transport system in Libya, which is still under construction. Its first two phases were completed in 1994 and 2000 respectively. The approximately 3500 km long system, which was made of concrete pipes of 4 metres in diameter, supplies about three million m^3/d of water. This is mainly used for irrigation and also partly for water supply of the cities in the coastal area of the country. After all the three remaining phases of construction have been completed, the total capacity provided will be approximately 5.7 million m^3/d. Figure 1.7 gives an impression of the size of the system by laying the territory of Libya (the grey area) over the map of Western Europe.

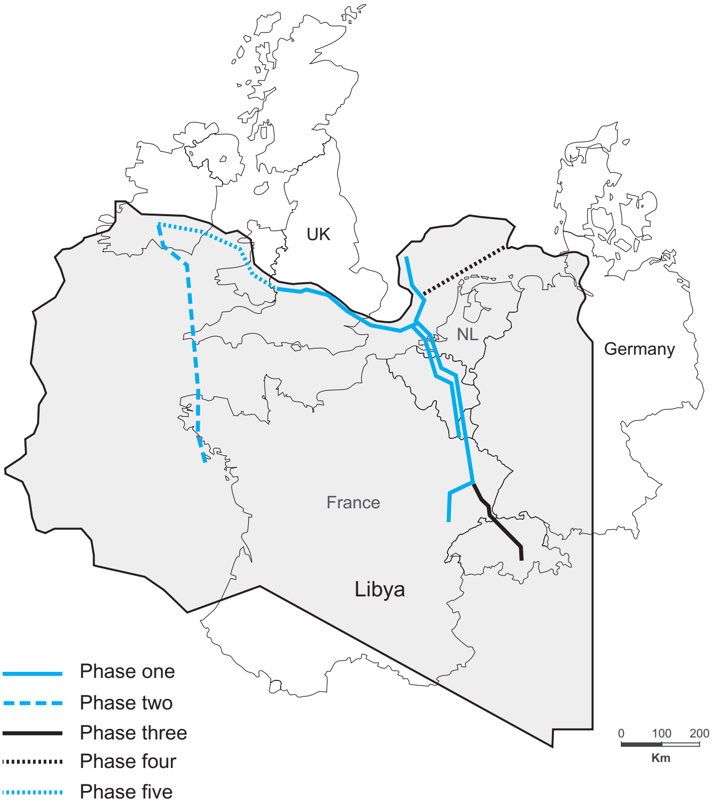


Figure 1.7. The 'Great Man-made River' transport system in Libya (The Management and Implementation Authority of the GMR project, 1989).

Water distribution systems

Water distribution systems consist of a network of smaller pipes with numerous connections that supply water directly to the users. The flow variations in such systems are much wider than in cases of water transport systems. In order to achieve optimal operation, different types of reservoirs, pumping stations, water towers, as well as various appurtenances (valves, hydrants, measuring equipment, etc.) can be installed in the system.

The example of a medium-size distribution system in Figure 1.8 shows the looped network of Zanzibar in Tanzania, a town of approximately 230,000 inhabitants. The average supply capacity is approximately 27,000 m^3/d (Hemed, 1996). Dotted lines in the figure indicate pipe routes planned for future extensions; the network layout originates from a computer model that consisted of some 200 pipes and was effectively used in describing the hydraulic performance of the network.

The main objectives of water transport and distribution systems are common:

– supply of adequate water quantities,
– maintaining the water quality achieved by the water treatment process.

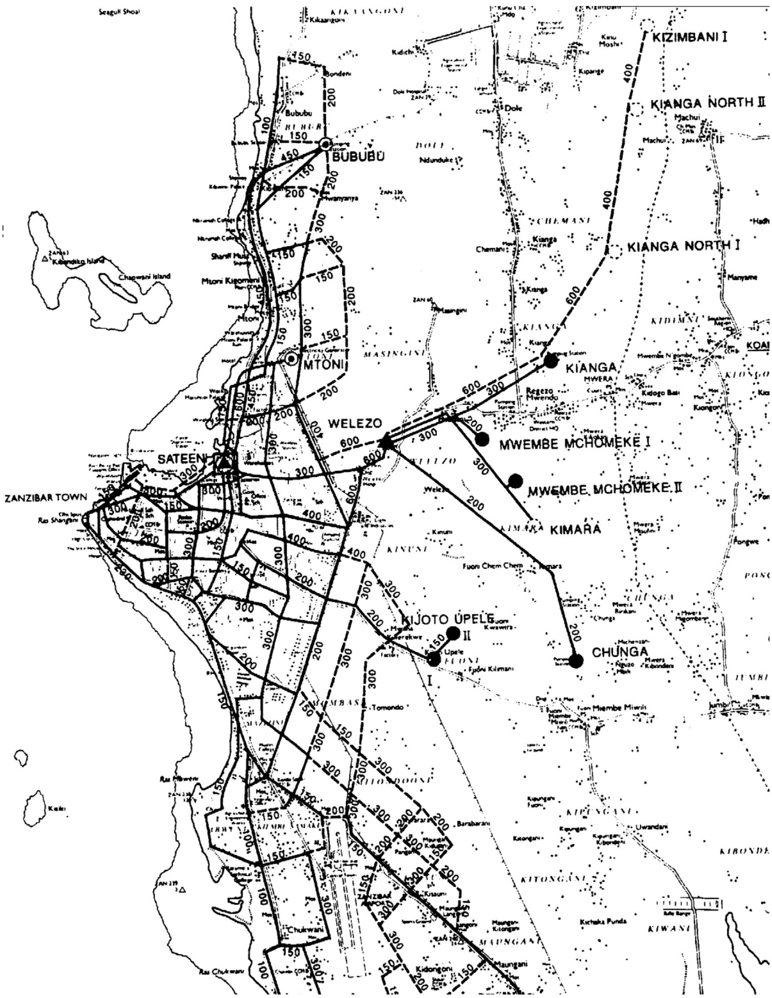


Figure 1.8. Water distribution system in Zanzibar, Tanzania (Hemed, 1996).

Each of these objectives should be satisfied for all consumers at any moment and, bearing in mind the massive scale of such systems, at an acceptable cost. This presumes a capacity of water supply for basic domestic purposes, commercial, industrial and other types of use and, where possible and economically justified, for fire protection.

Speaking in hydraulic terms, sufficient quantity and quality of water can be maintained by adequate pressure and velocity. Keeping pipes always under pressure drastically reduces the risks of external contamination. In addition, conveying the water at an acceptable velocity helps to reduce the retention times, which prevents the deterioration in quality resulting from low chlorine residuals, the appearance of sediments, the growth of micro organisms, etc. Hence, *potable water in transport and distribution systems must always be kept under a certain minimum pressure and for hygienic reasons should not be left stagnant in pipes*.

Considering the engineering aspects, the quantity and quality requirements are met by making proper choices in the selection of components and materials. System components used for water transport and distribution should be constructed i.e. manufactured from durable materials that are resistant to mechanical and chemical attacks, and at the same time not harmful for human health. Also importantly, their dimensions should comply with established standards.

Finally, in satisfying the quantity and quality objectives special attention should be paid to the level of workmanship during the construction phase as well as later on, when carrying out the system operation and maintenance. Lack of consistency in any of these indicated steps may result in the pump malfunctioning, leakages, bursts, etc. with the possible consequence of contaminated water.

1.2.2 *Piping*

Piping is a part of transport and distribution systems that demands major investments. The main components comprise pipes, joints, fittings, valves and service connections. According to the purpose they serve, the pipes can be classified as follows:

Trunk main

Trunk main is a pipe for the transport of potable water from treatment plant to the distribution area. Depending on the maximum capacity i.e. demand of the distribution area, the common range of pipe sizes is very wide; trunk mains can have diameters of between a few 100 millimetres and a few meters, in extreme cases. Some branching of the pipes is possible but consumer connections are rare.

Secondary mains

Secondary mains are pipes that form the basic skeleton of the distribution system. This skeleton normally links the main components, sources, reservoirs and pumping stations, and should enable the smooth distribution of bulk flows towards the areas of higher demand. It also supports the system operation under irregular conditions (fire, a major pipe burst or maintenance, etc.). A number of service connections can be provided from these pipes, especially for large consumers. Typical diameters are 150–400 mm.

Distribution mains

Distribution mains convey water from the secondary mains towards various consumers. These pipes are laid alongside roads and streets with numerous service connections and valves connected to guarantee the required level of supply. In principle, common diameters are between 80–200 mm.

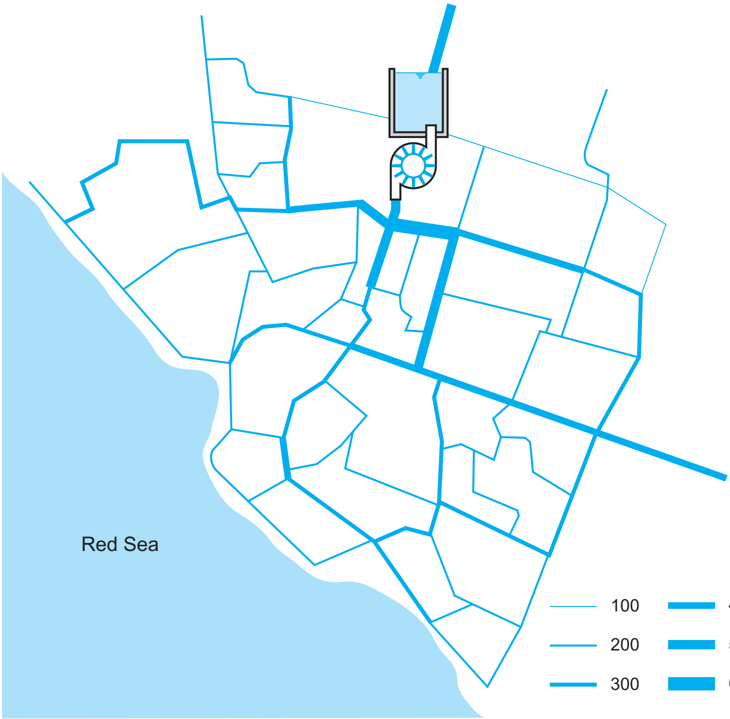


Figure 1.9. Distribution system in Hodaidah, Yemen (Trifunović and Blokland, 1993).

The schematic layout of a distribution network supplying some 350,000 consumers is given in Figure 1.9. The sketch shows the end of the trunk main that connects the reservoir and pumping station with the well field. The water is pumped from the reservoir through the network of secondary mains of diameters D = 300–600 mm and further distributed by the pipes D = 100 and 200 mm.

Service pipes

From the distribution mains, numerous *service pipes* bring the water directly to the consumers. In the case of domestic supplies, the service pipes are generally around 25 mm (1 inch) but other consumers may require a larger size.

The end of the service pipe is the end point of the distribution system. From that point on, two options are possible:

Public connection

Public connection; the service pipe terminates in one or more outlets and the water is consumed directly. This can be any type of public tap, fountain, etc.

Private connection

Private connection; the service pipe terminates at a stopcock of a private installation within a dwelling. This is the point where the responsibility

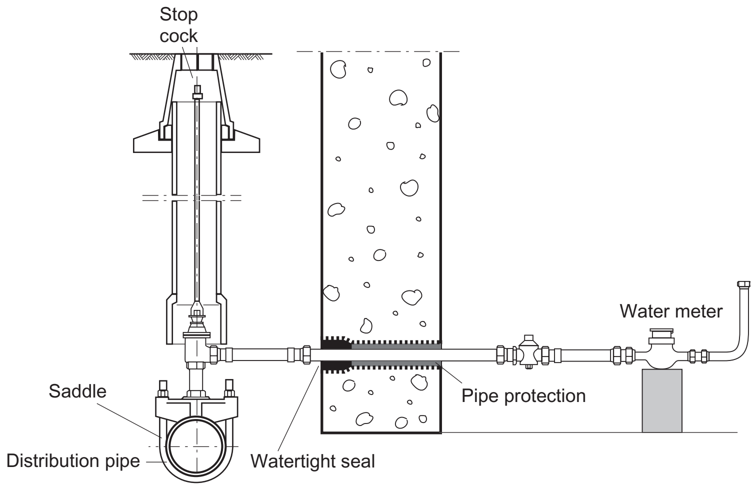


Figure 1.10. Schematic layout of a service connection.

of the water supply company usually stops. These can be different types of house or garden connections, as well as connections for non-domestic use.

One typical domestic service connection is shown in Figure 1.10.

1.2.3 *Storage*

Clear water storage facilities are a part of any sizable water supply system. They can be located at source (i.e. the treatment plant), at the end of the transport system or at any other favourable place in the distribution system, usually at higher elevations. Reservoirs (or tanks) serve the following general purposes:

- meeting variable supply to the network with constant water production,
- meeting variable demand in the network with its constant supply,
- providing a supply in emergency situations,
- maintaining stable pressure (if sufficiently elevated).

Except for very small systems, the costs of constructing and operating water storage facilities are comparable to the savings achieved in building and operating other parts of the distribution system. Without the use of a storage reservoir at the end of the transport system, the flow in the trunk main would have to match the demand in the distribution area at any moment, resulting in higher design flows i.e. larger pipe diameters. When operating in conjunction with the reservoir, this pipe only needs to be sufficient to convey the average flow, while the maximum peak flow is going to be supplied by drawing the additional requirement from the balancing volume.

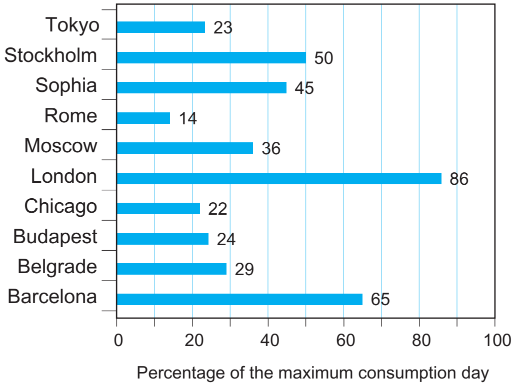


Figure 1.11. Total storage volume in some world cities (adapted from: Kujundžić, 1996).

Selection of an optimal site for a reservoir depends upon the type of supply scheme, topographical conditions, the pressure situation in the system, economical aspects, climatic conditions, security, etc. The required volume to meet the demand variations will depend on the daily demand pattern and the way the pumps are operated. Stable consumption over 24 hours normally results in smaller volume requirements than in cases where there is a big range between the minimum and maximum hourly demand. Finally, a proper assessment of needs for supply under irregular conditions can be a crucial decision factor.

Total storage volume in one distribution area commonly covers between 20–50% of the maximum daily consumption within any particular design year. With additional safety requirements, this percentage can be even higher. See Chapter 4 for a further discussion of the design principles. Figure 1.11 shows the total reservoir volumes in some world cities.

The reservoirs can be constructed either:

– underground,
– ground level or
– elevated (water towers).

Underground reservoirs are usually constructed in areas where safety or aesthetical issues are in question. In tropical climates, preserving the water temperature i.e. water quality could also be considered when choosing such a construction.

Compared to the underground reservoirs, the ground level reservoirs are generally cheaper and offer easier accessibility for maintenance. Both of these types have the same objectives: balancing demand and buffer reserve.

Water towers

Elevated tanks, also called *water towers*, are typical for predominantly flat terrains in cases where required pressure levels could not have been

Figure 1.12. Water towers in Amsterdam (still in use) and Delft (no longer in use).

reached by positioning the ground tank at some higher altitude. These tanks rarely serve as a buffer in irregular situations; large elevated volumes are generally unacceptable due to economical reasons. The role of elevated tanks is different compared to ordinary balancing or storage reservoirs. The volume here is primarily used for balancing of smaller and shorter demand variations and not for daily accumulation. Therefore, the water towers are often combined with pumping stations, preventing too frequent switching of the pumps and stabilising the pressure in the distribution area at the same time. Two examples of water towers are shown in Figure 1.12.

In some cases, tanks can be installed at the consumer's premises if:
- those consumers would otherwise cause large fluctuations of water demand,
- the fire hazard is too high,
- back-flow contamination of the distribution system (by the user) has to be prevented,
- an intermittent water supply is unavoidable.

In cases of restricted supply, individual storage facilities are inevitable. Very often, the construction of such facilities is out of proper control and the risk of contamination is relatively high. Nevertheless, in the absence of other viable alternatives, these are widely applied in arid areas of the world, such as in the Middle East, Southeast Asia or South America.

A typical example from Sana'a, the Republic of Yemen, in Figure 1.13 shows a ground level tank with a volume of 1–2 m^3, connected to the distribution network. This reservoir receives the water in periods when the pressure in the distribution system is sufficient. The

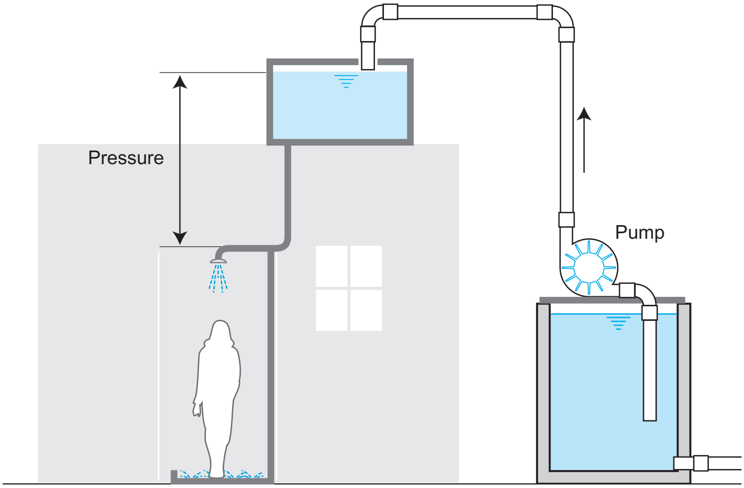


Figure 1.13. Individual storage in water scarce areas (Trifunović, 1994).

Figure 1.14. Roof tanks in Ramallah, Palestine (Trifunović, Abu-Madi, 1999).

pressure in the house installation is maintained from the roof tank that is filled by a small pump. Both reservoirs have float valves installed in order to prevent overflow. In more advanced applications, the pump may operate automatically depending on the water level in both tanks. In areas of the town with more favourable pressure, the roof tanks will be directly connected to the network (Figure 1.14).

In theory, this kind of supply allows for lower investment in distribution pipes as the individual balancing of demand reduces the peak flows in the system. In addition, generally lower pressures associated with the supply from the roof tanks affect leakages in a positive way. In practice however, the roof tanks are more often a consequence of a poor service level rather than a water demand management tool.

In Europe, roof tanks can be seen in arid areas of the Mediterranean belt. Furthermore, they are traditionally built in homes in the UK. The practice there dates from the nineteenth century when water supplied to homes from the municipal water companies was intermittent, which is the same reason as in many developing countries nowadays. Such tanks, usually of a few 100 litres, are typically installed under the roof of a family house and are carefully protected from external pollution. Their present role is now less for emergencies and more as small balancing tanks. Furthermore, the roof tanks in the developed world are frequently encountered in large multi-storey buildings, for provision of pressure and for fire fighting on the higher floors.

1.2.4 *Pumping*

Pumps add energy to water. Very often, the pumping operation is closely related to the functioning of the balancing reservoirs. Highly-elevated reservoirs will usually be located at the pressure (i.e. downstream) side of the pumping station in order to be refilled during the periods of low demand. The low-level reservoirs, on the other hand, will be positioned at the suction (i.e. upstream) side of the pumping station that provides supply to the consumers located at higher elevations. Apart from that, pumps can be located anywhere in the network where additional pressure is required (booster stations).

Centrifugal flow pumps

Centrifugal flow pumps are commonly used in water distribution. They can be installed in a horizontal or vertical set-up if available space is a matter of concern (see Figure 1.15). The main advantages of centrifugal pumps are low maintenance costs, high reliability, a long lifetime and simple construction, which all ensure that the water pumped is hygienically pure.

The pump unit is commonly driven by an electrical motor or a diesel engine, the latter being an alternative in case of electricity failures or in remote areas not connected at all to the electricity network. Two groups of pumps can be distinguished with respect to the motor operation:

1 fixed speed pumps,
2 variable speed pumps.

Frequency converter

In the first case, the pump is driven by a motor with a fixed number of revolutions. In the second case, an additional installed device, called the *frequency converter*, controls the impeller rotation enabling a more flexible pump operation.

Variable speed pumps

Variable speed pumps can achieve the same hydraulic effect as fixed speed pumps in combination with a water tower, rendering water towers unnecessary. By changing the speed, those pumps are able to follow the demand pattern within certain limits whilst at the same time keeping

Figure 1.15. Vertical centrifugal pump.

almost constant pressure. Consequently, the same range of flows can be covered with a smaller number of units. However, this technology has some restrictions; it cannot cover a large demand variation. Moreover, it involves rather sophisticated and expensive equipment, which is probably the reason why it is predominantly applied in the developed countries. With obvious cost-saving effects, variable speed pumps are widely used in The Netherlands where the vast majority of over 200 water towers built throughout the nineteenth and twentieth centuries has been disconnected from operation in recent years, being considered uneconomical.

Proper selection of the type and number of pump units is of crucial importance for the design of pumping stations. Connecting pumps in a parallel arrangement enables a wider range of flows to be covered by the pumping schedule while with pumps connected in serial arrangement the water can be brought to extremely high elevations. A good choice in both cases guarantees that excessive pumping heads will be minimised, pumping efficiency increased, energy consumption reduced, working hours of the pumps better distributed and their lifetime extended.

1.3 TYPES OF DISTRIBUTION SCHEMES

With respect to the way the water is supplied, the following distribution schemes can be distinguished:
1 gravity,
2 direct pumping,
3 combined.

The choice of one of the above alternatives is closely linked to the existing topographical conditions.

Gravity scheme

Gravity scheme makes use of the existing topography. The source is, in this case, located at a higher elevation than the distribution area itself. The water distribution can take place without pumping and nevertheless under acceptable pressure. The advantages of this scheme are:
– no energy costs,
– simple operation (fewer mechanical components, no power supply needed),
– low maintenance costs,
– slower pressure changes,
– a buffer capacity for irregular situations.

As much as they can help in creating pressure in the system, the topographical conditions may obstruct future supplies. Due to the fixed pressure range, the gravity systems are less flexible for extensions. Moreover, they require larger pipe diameters in order to minimise pressure losses. The main operational concern is capacity reduction that can be caused by air entrainment.

Direct pumping scheme

In the *direct pumping scheme*, the system operates without storage provision for demand balancing. The entire demand is directly pumped into the network. As the pumping schedule has to follow variations in water demand, the proper selection of units is important in order to optimise the energy consumption. Reserve pumping capacity for irregular situations should also be planned.

Advantages of the direct pumping scheme are opposite to those of the gravity scheme. With good design and operation, any pressure in the system can be reached. However, these are systems with rather complicated operation and maintenance and they are dependant on a reliable power supply. Additional precautions are therefore necessary, such as an alternative source of power supply, automatic mode of pump operation, stock of spare parts, etc.

Combined scheme

Combined scheme assumes an operation with pumping stations and demand balancing reservoirs. Part of the distribution area may be supplied by the direct pumping and the other part by gravity. A considerable storage volume is needed in this case but the pumping capacities will be below those in the direct pumping scheme. The combined systems are common in hilly distribution areas.

Pressure zones

The prevailing topography can lead to the use of *pressure zones*. By establishing different pressure zones, savings can be obtained in supplying water to the various elevations at lower pumping costs and in the use of lower-class piping due to the lower pressure. Technically, the pressure zones may be advantageous in preventing too high pressures in lower parts of the network (pressure-reducing valves may be used), or providing sufficient pressures in the higher parts (by pumping) when the source of supply is located in the lower zone.

An interesting application of zoning can be seen in Stuttgart, Germany. The distribution network for about 500,000 consumers is located in a terrain with an elevation difference of some 300 m between the lowest and the highest points. It is divided into nine pressure zones and in this way split into 56 small distribution areas. In each of these sub-areas the pressure range is kept between 20–60 metres water column (mwc). The main advantage of such a system is that a major failure is virtually impossible. In case of calamities in some of the sub-areas, an alternative supply from the neighbouring areas can be arranged by adjusting the pump operation. However, a centralised and very well synchronised control of the system is necessary to achieve this level of operation.

1.4 NETWORK CONFIGURATIONS

Depending on the way the pipes are interconnected, the following network configurations can be distinguished (see Figure 1.16):

1 serial,
2 branched,
3 grid (looped),
4 combined.

Serial network

Serial network is a network without branches or loops, the simplest configuration of all (Figure 1.16 A). It has one source, one end and a couple of intermediate nodes (demand points). Each intermediate node connects two pipes: supply i.e. an upstream pipe and distribution i.e. a downstream pipe. The flow direction is fixed from the source to the end of the system.

These networks characterise very small (rural) distribution areas and although rather cheap, they are not common due to extremely low reliability and quality problems caused by water stagnation at the end of the system. When this configuration is used for water transport, large diameters and lengths of the pipes will cause a drastic increase in the construction costs. Where reliability of supply is of greater concern than the construction cost, parallel lines will be laid.

Branched network

Branched network is a combination of serial networks. It usually consists of one supply point and several ends (Figure 1.16 B and C). In this case,

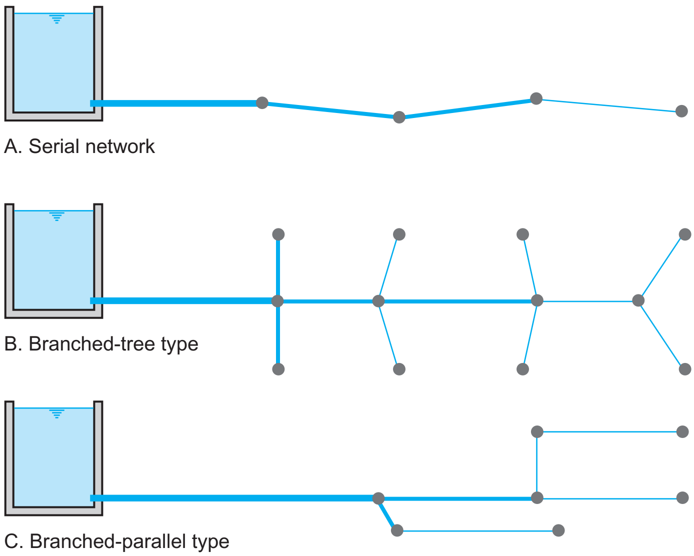


Figure 1.16. Serial and branched network configurations.

the intermediate nodes in the system connect one upstream pipe with one or several downstream pipes. Fixed flow direction is generated by the distribution from the source to the ends of the system.

Branched networks are adequate for small communities bearing in mind acceptable investment costs. However, the main disadvantages remain:

– low reliability,
– potential danger of contamination caused by large parts of the network being 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,
– a fluctuating water demand producing rather high pressure oscillations.

Grid systems

Grid systems (Figure 1.17 A and B) consist of nodes that can receive water from more than one side. This is a consequence of the looped structure of the network formed in order to eliminate the disadvantages of branched systems. Looped layout can be developed from a branched system by connecting its ends either at a later stage, or initially as a set of loops. The problems met in branched systems will be eliminated in the following circumstances:

– the water in the system flows in more than one direction and a long lasting stagnation does not easily occur any more,
– during the system maintenance, the area concerned will continue to be supplied by water flowing from other directions; in the case of pumped systems, a pressure increase caused by a restricted supply may even promote this,
– water demand fluctuations will produce less effect on pressure fluctuations.

Grid systems are hydraulically far more complicated than the serial or branched networks. The flow pattern in such a system is predetermined not only by its layout but also by the system operation. This means that the location of critical pressures may vary in time. In case of supply from more than one source, the analysis becomes even more complex.

Grid systems are more expensive both in investment and costs of operation. They are appropriate solutions predominantly for those (urban or industrial) distribution areas that require a high reliability of supply.

Combined network

Combined network is the most common type of network in large urban areas. The looped structure makes the central part of the system while

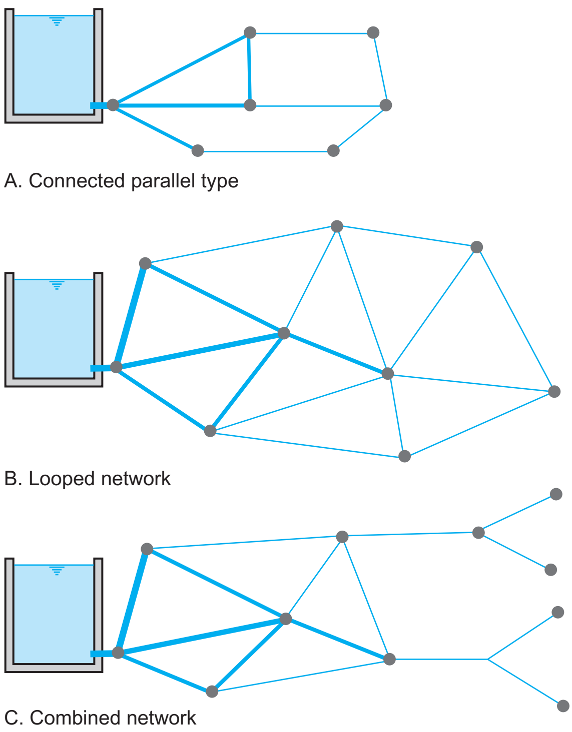


Figure 1.17. Looped network configurations.

the supply on the outskirts of the area is provided through a number of extended lines (Figure 1.17 C).

All the advantages and disadvantages of combined systems relating to both branched and looped systems have already been discussed.

CHAPTER 2

Water Demand

2.1 TERMINOLOGY

Water conveyance in a water supply system depends on the rates of production, delivery, consumption and leakage (Figure 2.1).

Water production

Water production (Q_{wp}) takes place at water treatment facilities. It normally has a constant rate that depends on the purification capacity of the treatment installation. The treated water ends up in a clear water reservoir from where it is supplied to the system (Reservoir A in Figure 2.1).

Water delivery

Water delivery (Q_{wd}) starts from the clear water reservoir of the treatment plant. Supplied directly to the distribution network, the generated flow will match certain demand patterns. When the distribution area is located far away from the treatment plant, the water is likely to be transported to another reservoir (B in Figure 2.1) that is usually constructed at the beginning of the distribution network. In principle, this delivery is done at the same constant flow rate that is equal to the water production.

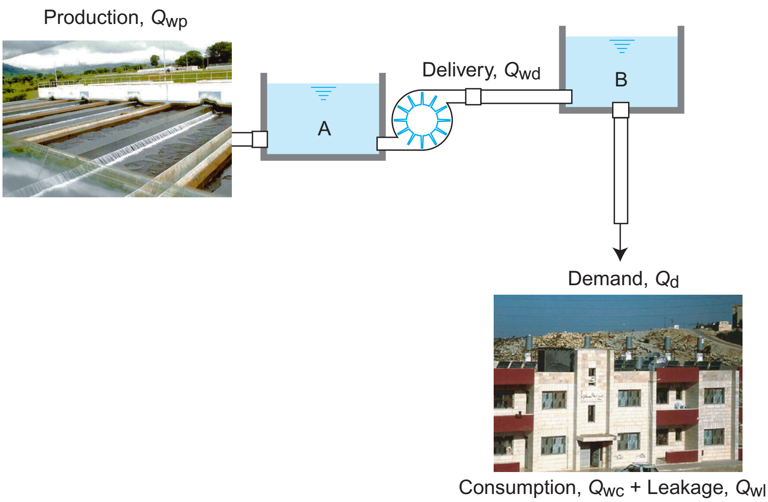


Figure 2.1. Flows in water supply systems.

Water consumption

Water consumption (Q_{wc}) is the quantity directly utilised by the consumers. This generates variable flows in the distribution network caused by many factors: users' needs, climate, source capacity etc.

Water leakage

Water leakage (Q_{wl}) is the amount of water physically lost from the system. The generated flow rate is in this case more or less constant and depends on overall conditions in the system.

Water demand

In theory, the term *water demand* (Q_d) coincides with water consumption. In practice, however, the demand is often monitored at supply points where the measurements include leakage, as well as the quantities used to refill the balancing tanks that may exist in the system. In order to avoid false conclusions, a clear distinction between the measurements at various points of the system should always be made. It is commonly agreed that $Q_d = Q_{wc} + Q_{wl}$. Furthermore, when supply is calculated without having an interim water storage, i.e. water goes directly to the distribution network: $Q_{wd} = Q_d$, otherwise: $Q_{wd} = Q_{wp}$.

Water demand is commonly expressed in cubic meters per hour (m^3/h) or per second (m^3/s), litres per second (l/s), mega litres per day (Ml/d) or litres per capita per day (l/c/d or lpcpd). Typical Imperial units are cubic feet per second (ft^3/s), gallon per minute (gpm) or mega gallon per day (mgd).[1] The mean value derived from annual demand records represents the *average demand*. Divided by the number of consumers, the average demand becomes the *specific demand (unit consumption per capita)*.

Average demand
Specific demand

Apart from neglecting leakage, the demand figures can often be misinterpreted due to lack of information regarding the consumption of various categories. Table 2.1 shows the difference in the level of specific demand depending on what is, or is not, included in the figure. The last two groups in the table coincide with commercial and domestic water use, respectively.

Table 2.1. Water demand in The Netherlands in 2001 (VEWIN).

	Annual (10^6 m^3)	Q_d(l/c/d)[1]
Total water delivered by water companies	1247	214
Drinking water delivered by water companies	1177	202
Drinking water paid for by consumers	1119	192
Consumers below 10,000 m^3/y per connection (metered)	940	161
Consumers below 300 m^3/y per connection (metered)	714	122

[1] Based on total population of approx. 16 million.

[1] A general unit conversion table is given in Appendix 7. See also spreadsheet lesson A5.8.1: 'Flow Conversion' (Appendix 5).

Accurate forecasting of water demand is crucial whilst analysing the hydraulic performance of water distribution systems. Numerous factors affecting the demand are determined from the answers to three basic questions:

1 *For which purpose is the water used?* The demand is affected by a number of consumption categories: domestic, industrial, tourism etc.
2 *Who is the user?* Water use within the same category may vary due to different cultures, education, age, climate, religion, technological process etc.
3 *How valuable is the water?* The water may be used under circumstances that restrict the demand: scarce source (quantity/quality), bad access (no direct connection, fetching from a distance), low income of consumers etc.

Answers to the above questions reflect on the quantities and moments when the water will be used, resulting in a variety of demand patterns. Analysing or predicting these patterns is not always an easy task. Uncritical adoption of other experiences where the field information is lacking is the wrong approach; each case is independent and the conclusions drawn are only valid for local conditions.

Variations in water demand are particularly visible in developing countries where prosperity is predominantly concentrated in a few major, usually overcrowded, cities with peripheral areas often having restricted access to drinking water. These parts of the system will be supplied from public standpipes, individual wells or tankers, which cause substantial differences in consumption levels within the same distribution area. Figure 2.2 shows average specific consumption for a number of large cities in Asia.

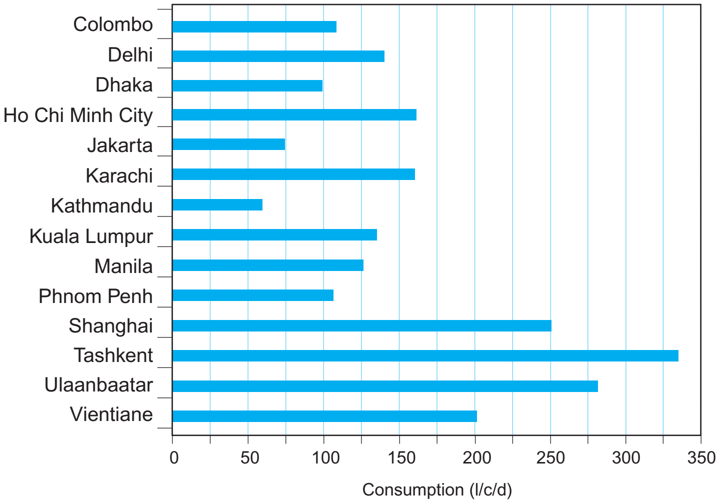


Figure 2.2. Specific consumption in Asian cities (McIntosh, 2003).

Table 2.2. Specific demand around Lake Victoria in Africa (IIED, 2000).

	Piped (l/c/d)	Un-piped (l/c/d)
Average for the entire region	45	22
Average for urban areas (small towns)	65	26
Average for rural areas	59	8
Part of the region in Uganda	44	19
Part of the region in Tanzania	60	24
Part of the region in Kenya	57	21

Comparative figures in Africa are generally lower, resulting from the range of problems that cause intermediate supply, namely long distances, electricity failures, pipe bursts, polluted ground water in deep wells, etc.

A water demand survey was conducted for the region around Lake Victoria, covering parts of Uganda, Tanzania and Kenya. The demand where there is a piped supply (the water is tapped at home) was compared with the demand in un-piped systems (no house connection is available). The results are shown in Table 2.2.

Unaccounted-for water

An unavoidable component of water demand is *unaccounted-for water* (UFW), the water that is supplied 'free of charge'. In quite a lot of transport and distribution systems in developing countries this is the most significant 'consumer' of water, accounting sometimes for over 50% of the total water delivery.

Causes of UFW differ from case to case. Most often it is a leakage that appears due to improper maintenance of the network. Other non-physical losses are related to the water that is supplied and has reached the taps, but is not registered or paid for (under-reading of water meters, illegal connections, washing streets, flushing pipes, etc.)

2.2 CONSUMPTION CATEGORIES

2.2.1 *Water use by various sectors*

Water consumption is initially split into domestic and non-domestic components. The bulk of non-domestic consumption relates to the water used for agriculture, occasionally delivered from integral water supply systems, and for industry and other commercial uses (shops, offices, schools, hospitals, etc.). The ratio between the domestic and non-domestic consumption in The Netherlands in the period 1960–2000 is shown in Figure 2.3.[2]

[2] The domestic consumption in Figure 2.3 is derived from consumers metered below 300 m^3/y per connection. The real consumption is assumed to be slightly higher; the figure assessed by VEWIN for 2001 is 126 l/c/d compared to 134 l/c/d in 1995.

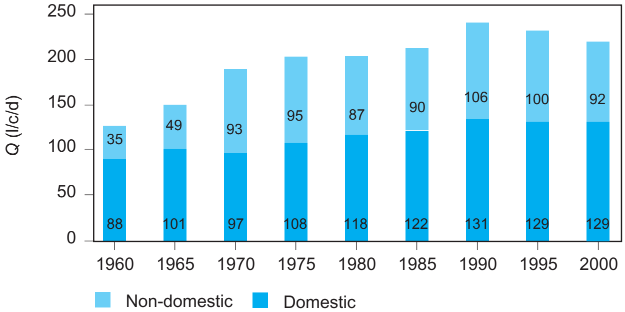


Figure 2.3. Domestic and non-domestic consumption in The Netherlands (VEWIN).

Table 2.3. Domestic vs. non-domestic consumption in some African states (SADC, 1999).

Country	Agriculture (%)	Industry (%)	Domestic (%)
Angola	76	10	14
Botswana	48	20	32
Lesotho	56	22	22
Malawi	86	3	10
Mozambique	89	2	9
South Africa	62	21	17
Zambia	77	7	16
Zimbabwe	79	7	14

In the majority of developing countries, agricultural- and domestic water consumption is predominant compared to the commercial water use, as the example in Table 2.3 shows. However, this water is rarely supplied from an integral system.

In warm climates, the water used for irrigation is generally the major component of total consumption; Figure 2.4 shows an example of some European countries around the Mediterranean Sea: Spain, Italy and Greece. On the other hand, highly industrialised countries use huge quantities of water, often of drinking quality, for cooling; typical examples are Germany, France and Finland, which all use more than 50% of the total consumption for this purpose. Striving for more efficient irrigation methods, industrial processes using alternative sources and recycling water have been and still are a concern in developed countries for the last few decades.

2.2.2 *Domestic consumption*

Domestic water consumption is intended for toilet flushing, bathing and showering, laundry, dishwashing and other less water intensive or less frequent purposes: cooking, drinking, gardening, car washing, etc. The

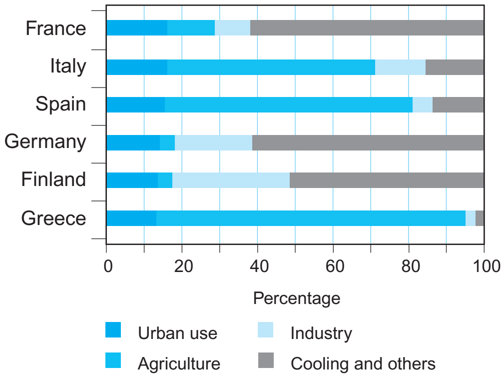


Figure 2.4. Water use in Europe (EEA, 1999).

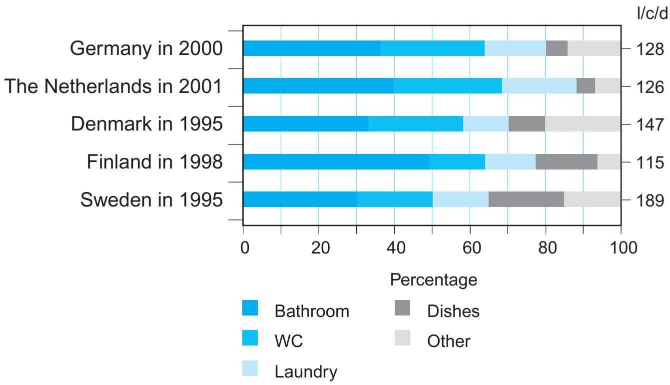


Figure 2.5. Domestic water use in Europe (EEA, BGW, VEWIN).

example in Figure 2.5 shows rather wide variation in the average domestic consumption of some industrialised countries. Nevertheless, in all the cases indicated 50–80% of the total consumption appears to be utilised in bathrooms and toilets.

The habits of different population groups with respect to water use were studied in The Netherlands (Achttienribbe, 1993). Four factors compared were age, income level, household size and region of the country. The results are shown in Figure 2.6.

The figures prove that even with detailed statistics available, conclusions about global trends may be difficult. In general, the consumption is lower in the northern part of the country, which is a less populated, mostly agricultural region. Nonetheless, interesting findings from the graphs are evident: the middle-aged group is the most moderate water user, more frequent toilet use and less frequent shower use is exercised by older groups, larger families are with a lower consumption per capita, etc.

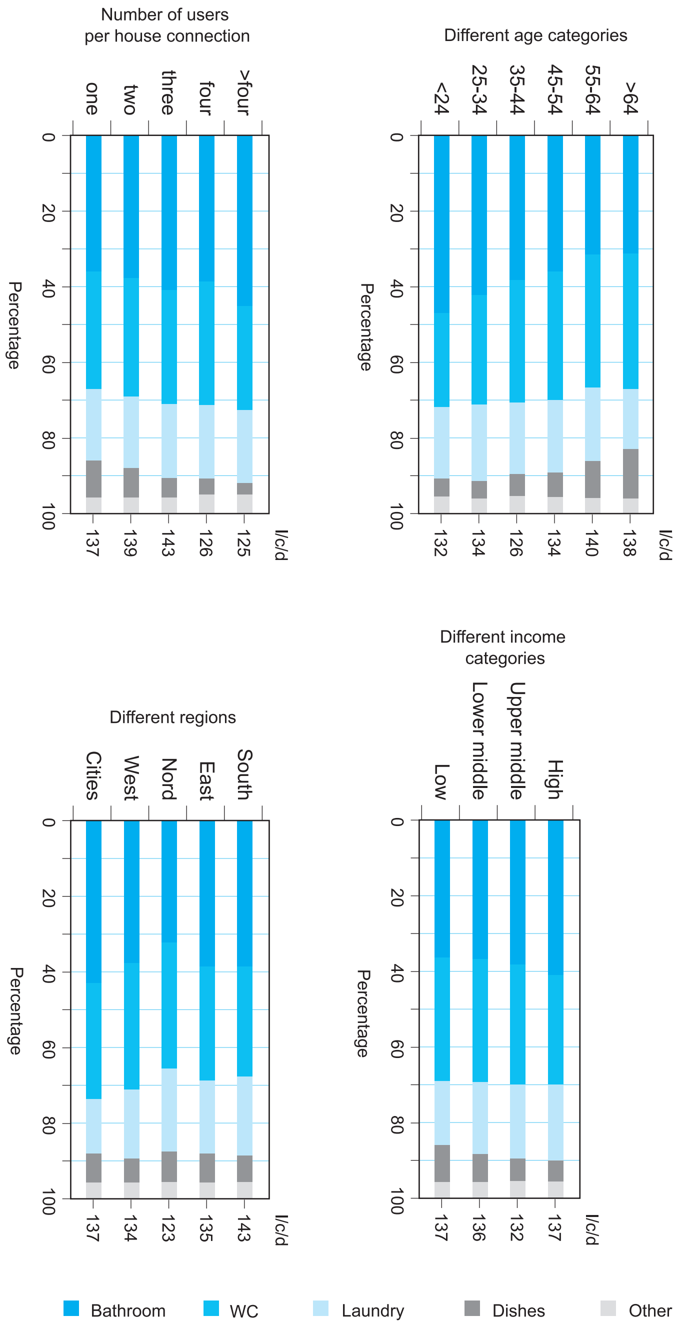


Figure 2.6. Structure of domestic consumption in The Netherlands (Achttienribbe, 1993).

In cases where there is an individual connection to the system, the structure of domestic consumption in water scarce areas may well look similar but the quantity of water used for particular activities will be minimised. Apart from the change of habits, this is also a consequence of low pressures in the system directly affecting the quantities used for showering, gardening, car washing, etc. On top of this, the water company may be forced to ration the supply by introducing regular interruptions. In these situations consumers will normally react by constructing individual tanks. In urban areas where supply with individual tanks takes place, the amounts of water available commonly vary between 50–100 l/c/d.

2.2.3 *Non-domestic consumption*

Non-domestic or commercial water use occurs in industry, agriculture, institutions and offices, tourism, etc. Each of these categories has its specific water requirements.

Industry
Water in industry can be used for various purposes: as a part of the final product, for the maintenance of manufacturing processes (cleaning, flushing, sterilisation, conveying, cooling, etc) and for the personal needs (usually comparatively marginal). The total quantities will largely depend on the type of industry and technological process. They are commonly expressed in litres per unit of product or raw material. Table 2.4 gives an indication for a number of industries; an extensive overview can be found in HR Wallingford (2003).

Table 2.4. Industrial water consumption (Adapted from: HR Wallingford, 2003).

Industry	Litres per unit product
Carbonated soft drinks[1]	1.5–5 per litre
Fruit juices[1]	3–15 per litre
Beer[1]	4–22 per litre
Wine	1–4 per litre
Fresh meat (red)	1.5–9 per kg
Canned vegetables/fruits	2–27 per kg
Bricks	15–30 per kg
Cement	4 per kg
Polyethylene	2.5–10 per kg
Paper[2]	4–35 per kg
Textiles	100–300 per kg
Cars	2500–8000 per car

Notes
[1] Largely dependant on the packaging and cleaning of bottles.
[2] Recycled paper.

Agriculture

Water consumption in agriculture is mainly determined by irrigation and livestock needs. In peri-urban or developed rural areas, this demand may also be supplied from the local distribution system.

The amounts required for irrigation purposes depend on the plant sort, stage of growth, type of irrigation, soil characteristics, climatic conditions, etc. These quantities can be assessed either from records or by simple measurements. A number of methods are available in literature to calculate the consumption based on meteorological data (Blaney-Criddle, Penman, etc.). According to Brouwer and Heibloem (1986), the consumption is unlikely to exceed a monthly mean of 15 mm per day, which is equivalent to 150 m^3/d per hectare. Approximate values per crop are given in Table 2.5.

Water required for livestock depends on the sort and age of the animal, as well as climatic conditions. Size of the stock and type of production also play a role. For example, the water consumption for milking cows is 120–150 l/d per animal, whilst cows typically need only 25 l/d (Brandon, 1984) (see Table 2.6).

Table 2.5. Seasonal crop water needs (Brouwer and Heibloem, 1986).

Crop	Season (days/year)	Consumption (mm/season)
Bananas	300–365	1200–2200
Beans	75–110	300–500
Cabbages	120–140	350–500
Citrus fruit	240–365	900–1200
Corn	80–180	500–800
Potatoes	105–145	500–700
Rice	90–150	450–750
Sunflowers	125–130	600–1000
Tomatoes	135–180	400–800
Wheat	120–150	450–650

Table 2.6. Animal water consumption (Brandon, 1984).

Animal	Litres per day
Cows	25–150
Oxen, horses, etc.	15–40
Pigs	10–30
Sheep, goats	5–6
Turkeys (per 100)	65–70
Chickens (per 100)	25–30
Camels	2–3

Institutions

Commercial consumption in restaurants, shops, schools and other institutions can be assessed as a total supply divided by the number of consumers (employees, pupils, patients, etc.). Accurate figures should be available from local records at water supply companies. Some indications of unit consumption are given in Table 2.7. These assume individual connection with indoor water installations and waterborne sanitation, and are only relevant during working days.

Tourism

Tourist and recreational activities may also have a considerable impact on water demand. The quantities per person (or per bed) per day vary enormously depending on the type and category of accommodation; in luxury hotels, for instance, this demand can go up to 600 l/c/d. Table 2.8 shows average figures in Southwest England.

Miscellaneous groups

Water consumption that does not belong to any of the above-listed groups can be classified as miscellaneous. These are the quantities used for fire fighting, public purposes (washing streets, maintaining green areas, supply for fountains, etc.), maintenance of water and sewage systems (cleansing, flushing mains) or other specific uses (military facilities, sport complexes, zoos, etc.). Sufficient information on water consumption in such cases should be available from local records.

Table 2.7. Water consumption in institutions (adapted from: HR Wallingford, 2003).

Premises	Consumption
Schools	25–75 l/d per pupil
Hospitals	350–500 l/d per bed
Laundries	8[1]–60 litre per kg washing
Small businesses	25 l/d per employee
Retail shops/stores	100–135 l/d per employee
Offices	65 l/d per employee

[1] Recycled water used for rinsing

Table 2.8. Tourist water consumption in Southwest England (Brandon, 1984).

Accommodation	Consumption (l/c/d)
Camping sites	68
Unclassified hotels	113
Guest houses	130
1- and 2-star hotels	168
3-, 4- and 5-star hotels	269

Sometimes this demand is unpredictable and can only be estimated on an empirical or statistical basis. For example, in the case of fire fighting, the water use is not recorded and measurements are difficult because it is not known in advance when and where the water will be needed. Provision for this purpose will be planned with respect to potential risks, which is a matter discussion between the municipality (fire department) and water company.

On average, these consumers do not contribute substantially in overall demand. Very often they are neither metered nor accounted for and thus classified as UFW.

PROBLEM 2.1
A water supply company has delivered an annual quantity of 80,000,000 m^3 to a city of 1.2 million inhabitants. Find out the specific demand in the distribution area. In addition, calculate the domestic consumption per capita with leakage from the system estimated at 15% of the total supply, and billed non-domestic consumption of 20,000,000 m^3/y.

Answer:
Gross specific demand can be determined as:

$$Q_{\text{avg}} = \frac{80{,}000{,}000 \times 1000}{1{,}200{,}000/365} \approx 183 \text{ l/c/d}$$

The leakage of 15% of the total supply amounts to an annual loss of 12 million m^3. Reducing the total figure further for the registered non-domestic consumption yields the annual domestic consumption of $80-12-20 = 48$ million m^3, which is equal to a specific domestic consumption of approx.110 l/c/d.

Self-study:
Workshop problems A1.1.1 and A1.1.2 (Appendix 1)
Spreadsheet lesson A5.8.1 (Appendix 5)

2.3 WATER DEMAND PATTERNS

Each consumption category can be considered not only from the perspective of its average quantities but also with respect to the timetable of when the water is used.

Demand variations are commonly described by the *peak factors*. These are the ratios between the demand at particular moments and the average demand for the observed period (hour, day, week, year, etc.). For example, if the demand registered during a particular hour was 150 m^3

and for the whole day (24 hours) the total demand was 3000 m^3, the average hourly demand of 3000/24 = 125 m^3 would be used to determine the peak factor for the hour, which would be 150/125 = 1.2. Other ways of peak demand representation are either as a percentage of the total demand within a particular period (150 m^3 for the above hour is equal to 5% of the total daily demand of 3000 m^3), or simply as the unit volume per hour (150 m^3/h).

Human activities have periodic characteristics and the same applies to water use. Hence, the average water quantities from the previous paragraph are just indications of total requirements. Equally relevant for the design of water supply systems are consumption peaks that appear during one day, week or year. A combination of these maximum and minimum demands defines the absolute range of flows that are to be delivered by the water company.

Time-wise, we can distinguish the *instantaneous, daily (diurnal), weekly* and *annual (seasonal)* pattern in various areas (home, building, district, town, etc.). The larger the area is, the more diverse the demand pattern will be as it then represents a combination of several consumption categories, including leakage.

2.3.1 *Instantaneous demand*

Simultaneous demand

Instantaneous demand (in some literature *simultaneous demand*) is caused by a small number of consumers during a short period of time: a few seconds or minutes. Assessing this sort of demand is the starting point in building-up the demand pattern of any distribution area. On top of that, the instantaneous demand is directly relevant for network design in small residential areas (tertiary networks and house installations). The demand patterns of such areas are much more unpredictable than the demand patterns generated by larger number of consumers. *The smaller the number of consumers involved, the less predictable the demand pattern will be*.

The following *hypothetical* example illustrates the relation between the peak demands and the number of consumers.

A list of typical domestic water activities with provisional unit quantities utilised during a particular period of time is shown in Table 2.9. Parameter Q_{ins} in the table represents the average flow obtained by dividing the total quantity with the duration of the activity, converted into litres per hour.

Instantaneous flow

For example, activity 'A–Toilet flushing' is in fact refilling of the toilet cistern. In this case there is a volume of 8 l, within say one minute after the toilet has been flushed. In theory, to be able to fulfil this requirement, the pipe that supplies the cistern should allow the flow of $8 \times 60 = 480$ l/h within one minute. This flow is thus needed within a

Table 2.9. Example of domestic unit water consumption.

Activity	Total quantity (litres)	Duration (minutes)	Q_{ins} (l/h)
A – Toilet flushing	8	1	480
B – Showering	50	6	500
C – Hand washing	2	1/2	240
D – Face and teeth	3	1	180
E – Laundry	60	6	600
F – Cooking	15	5	180
G – Dishes	40	6	400
H – Drinking	1/4	1/20	300
I – Other	5	2	150

relatively short period of time and is therefore called the *instantaneous flow*.

Although the exact moment of water use is normally unpredictable, it is well known that there are some periods of the day when it happens more frequently. For most people this is in the morning after they wake-up, in the afternoon when they return from work or school or in the evening before they go to sleep.

Considering a single housing unit, it is not reasonable to assume a situation in which all water-related activities from the above table are executed simultaneously. For example, in the morning, a combination of activities A, B, D and H might be possible. If this is the assumed maximum demand during the day, the maximum instantaneous flow equals the sum of the flows for these four activities. Hence, the pipe that provides water for the house has to be sufficiently large to convey the flow of:

$$480 + 500 + 180 + 300 = 1460 \text{ l/h}$$

Instantaneous peak factor

With an assumed specific consumption of 120 l/c/d and, say, four people living together, the *instantaneous peak factor* will be:

$$pf_{ins} = \frac{1460}{120 \times 4/24} = 73$$

Thus, there was at least one short moment within 24 hours when the instantaneous flow to the house was 73 times higher than the average flow of the day.

Applying the same logic for an apartment building, one can assume that all tenants use the water there in a similar way and at a similar moment, but never in exactly the same way *and* at exactly the same moment. Again, the maximum demand of the building occurs in the morning. This could consist of, for example, toilet flushing in say three apartments, hand washing in two, teeth brushing in six, doing the laundry in two and drinking water in one. The maximum instantaneous flow out of such a consumption scenario case would be:

$$3A + 3B + 2C + 6D + 2E + 1H = 6000\ \text{l/h}$$

which is the capacity that has to be provided by the pipe that supplies the building. Assuming the same specific demand of 120 l/c/d and for possibly 40 occupants, the instantaneous peak factor is:

$$pf_{\text{ins}} = \frac{6000}{120 \times 40/24} = 30$$

Any further increase in the number of consumers will cause the further lowering of the instantaneous peak factor, up to a level where this factor becomes independent from the growth in the number of consumers. As a consequence, some large diameter pipes that have to convey water for possibly 100,000 consumers would probably be designed based on a rather low instantaneous peak factor, which in this example could be 1.4.

Simultaneity diagram

A *simultaneity diagram* can be obtained by plotting the instantaneous peak factors against the corresponding number of consumers. The three points from the above example, interpolated exponentially, will yield the graph shown in Figure 2.7.

In practice, the simultaneity diagrams are determined from a field study for each particular area (town, region or country). Sometimes, a good approximation is achieved by applying mathematical formulae; the equation: $pf_{\text{ins}} \approx 126 \times e^{(-0.9 \times \log N)}$ where N represents the number of consumers, describes the curve in Figure 2.7. Furthermore, the simultaneous

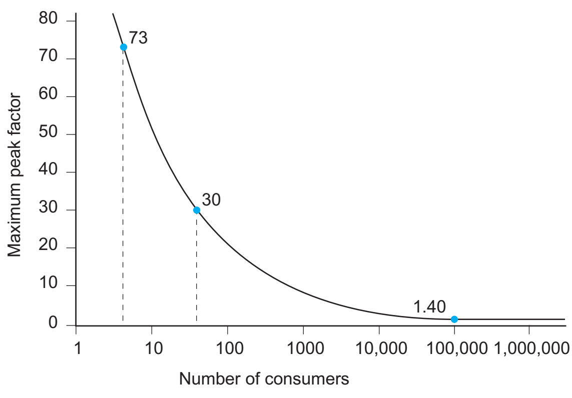


Figure 2.7. Simultaneity diagram (example).

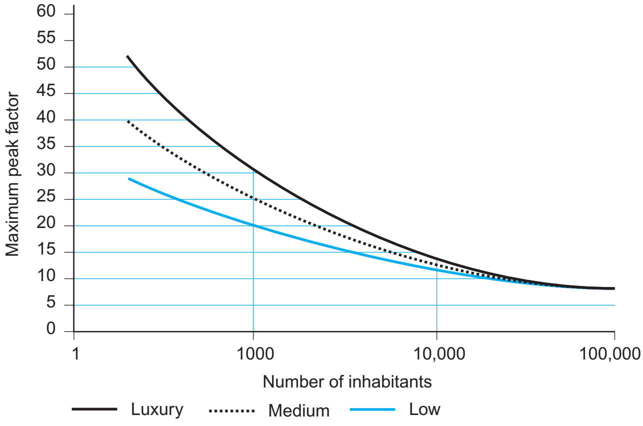


Figure 2.8. Simultaneity diagram of various categories of accommodation.

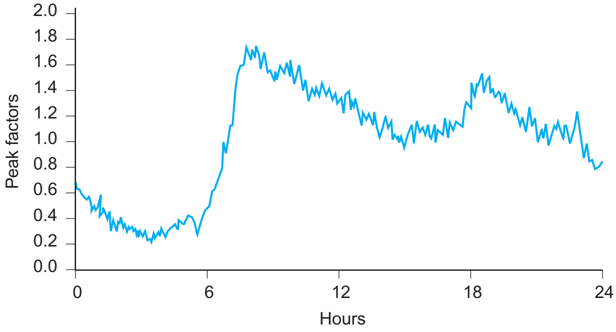


Figure 2.9. Demand pattern in Amsterdam (Municipal Water Company Amsterdam, 2002).

curves can be diversified based on various standards of living i.e. type of accommodation, as Figure 2.8 shows.

In most cases, the demand patterns of more than a few thousand people are fairly predictable. This eventually leads to the conclusion that the water demand of larger group of consumers will, in principle, be evenly spread over a period of time that is longer than a few seconds or minutes. This is illustrated in the 24-hour demand diagram shown in Figure 2.9 for the northern part of Amsterdam. In this example there were nearly 130,000 consumers, and the measurements were executed at 1-minute intervals.

Hourly peak factor

One-hour durations are commonly accepted for practical purposes and the instantaneous peak factor within this period of time will be represented by a single value called the *hourly (or diurnal) peak factor*, as shown in Figure 2.10.

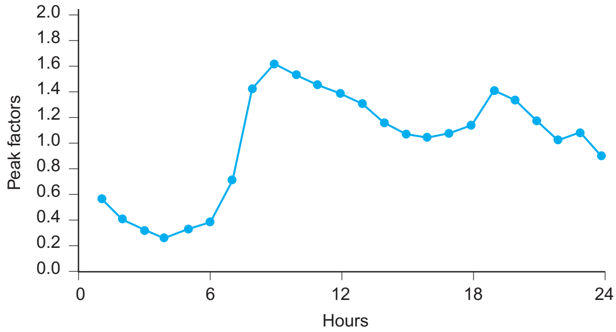


Figure 2.10. Instantaneous demand from Figure 2.9 averaged by the hourly peak factors.

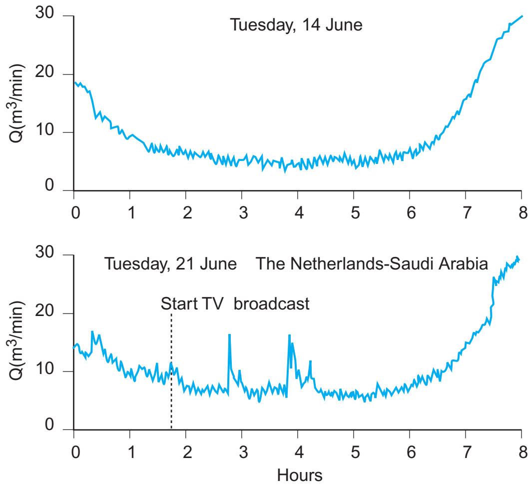


Figure 2.11. Night-time demand during football game (Water Company 'N-W Brabant', NL, 1994).

There are however extraordinary situations when the instantaneous demand may substantially influence the demand pattern, even in the case of large numbers of consumers.

Figures 2.11 and 2.12 show the demand pattern (in m^3/min) during the TV broadcasting of two football matches when the Dutch national team played against Saudi Arabia and Belgium at the 1994 World Cup in the United States of America. The demand was observed in a distribution area of approximately 135,000 people.

The excitement of the viewers is clearly confirmed through the increased water use during the break and at the end of the game, despite the fact that the first match was played in the middle of the night (with different time zones between The Netherlands and USA). Both graphs point almost precisely to the start of the TV broadcast that happened at 01:50 and 18:50, respectively. The water demand dropped soon after the start of the game until the half time when the first peak occurs; it is not difficult to guess for what purpose the water was used! The upper curves

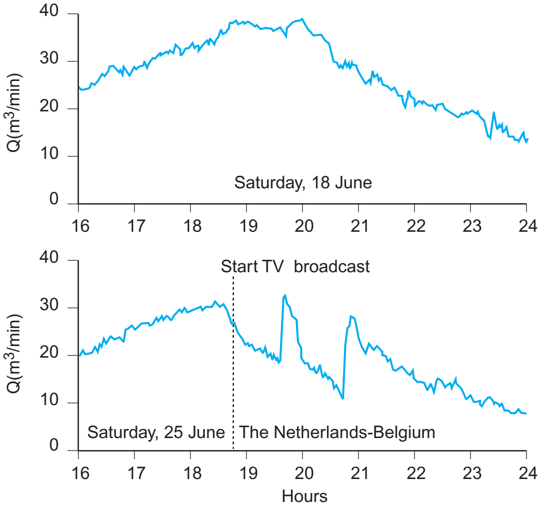


Figure 2.12. Evening demand during football game (Water Company 'N-W Brabant', NL, 1994).

in both figures show the demand under normal conditions, one week before the game at the same period of the day.

This phenomenon is not only typical in The Netherlands; it will be met virtually everywhere where football is sufficiently popular. Its consequence is a temporary drop of pressure in the system while in the most extreme situations a pump failure might occur. Nevertheless, these demand peaks are rarely considered as design parameters and adjusting operational settings of the pumps can easily solve this problem.

PROBLEM 2.2

In a residential area of 10,000 inhabitants, the specific water demand is estimated at 100 l/c/d (leakage included). During a football game shown on the local TV station, the water meter in the area registered the maximum flow of 24 l/s, which was 60% above the regular use for that period of the day. What was the instantaneous peak factor in that case? What would be the regular peak factor on a day without a televised football broadcast?

Answers:

In order to calculate the peak factors, the average demand in the area has to be brought to the same units as the peak flows. Thus, the average flow becomes:

$$Q_{avg} = \frac{10{,}000 \times 100}{24/3600} \approx 12 \text{ l/s}$$

The regular peak flow at a particular point of the day is 60% lower than the one registered during the football game, which is 24/1.6 = 15 l/s.

Finally, the corresponding peak factors will be 24/12 = 2 during the football game, and 15/12 = 1.25 in normal supply situations.

Self-study:
Workshop problems A1.1.3–A1.1.5 (Appendix 1)

2.3.2 *Diurnal patterns*

Diurnal demand diagram

For sufficiently large group of consumers, the instantaneous demand pattern for 24-hour period converts into a *diurnal (daily) demand diagram*. Diurnal diagrams are important for the design of primary and secondary networks, and in particular their reservoirs and pumping stations. Being the shortest cycle of water use, a one-day period implies a synchronised operation of the system components with similar supply conditions occurring every 24 hours.

The demand patterns are usually registered by monitoring flows at delivery points (treatment plants) or points in the network (pressure boosting stations, reservoirs, control points with either permanent or temporary measuring equipment). With properly organised measurements the patterns can also be observed at the consumers' premises. First, such an approach allows the separation of various consumption categories and second, the leakage in the distribution system will be excluded, resulting in a genuine consumption pattern.

A few examples of diagrams for different daily demand categories are given in Figures 2.13–2.16.

A flat daily demand pattern reflects the combination of impacts from the following factors:

- large distribution area,
- high industrial demand,
- high leakage level,
- scarce supply (individual storage).

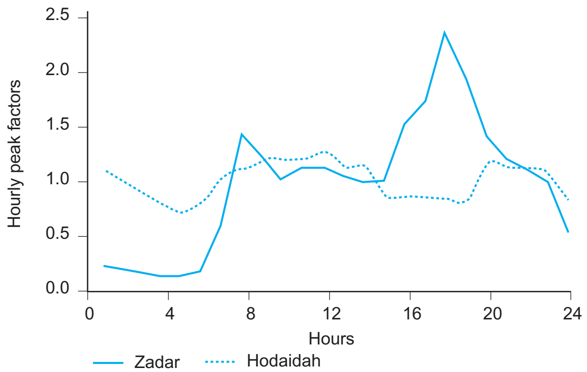


Figure 2.13. Urban demand pattern (adapted from: Gabrić, 1996 and Trifunović, 1993).

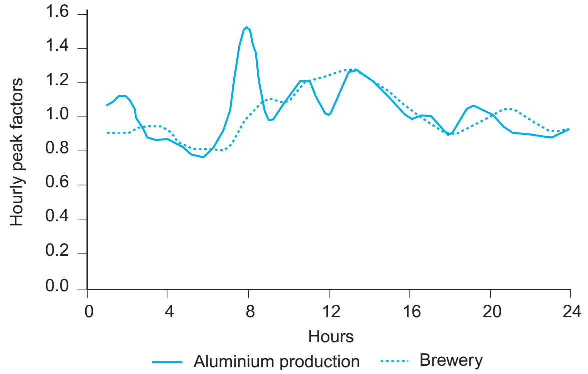


Figure 2.14. Industrial demand pattern – example from Bosnia and Herzegovina (Obradović, 1991).

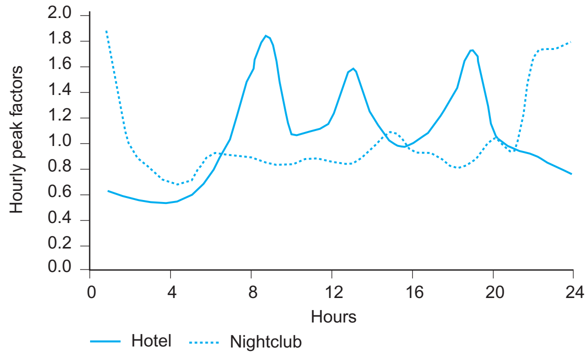


Figure 2.15. Tourist demand pattern – example from Croatia (Obradović, 1991).

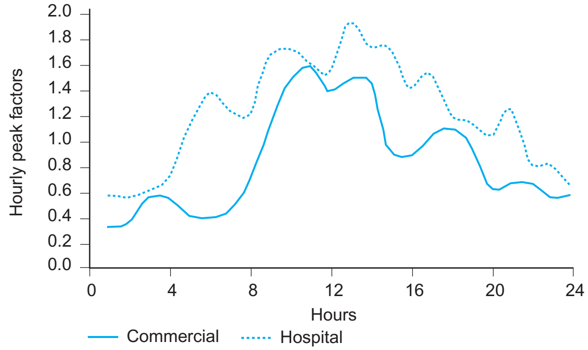


Figure 2.16. Commercial/ institutional demand pattern – example from USA (Obradović, 1991).

Commonly, the structure of the demand pattern in urban areas looks as shown in Figure 2.17: the domestic category will have the most visible variation of consumption throughout the day, industry and institutions will usually work in daily shifts, and the remaining categories, including leakage, are practically constant.

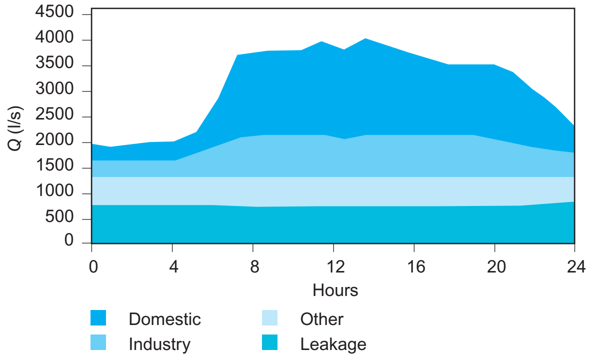


Figure 2.17. Typical structure of diurnal demand in urban areas.

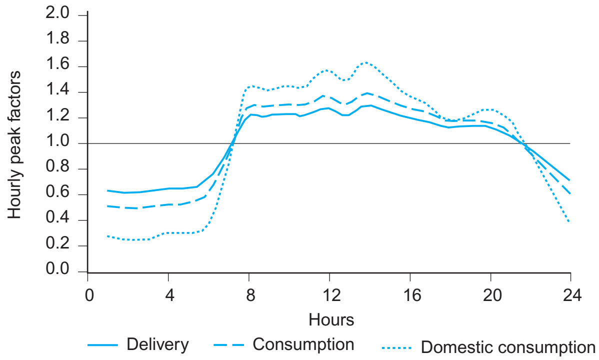


Figure 2.18. Peak factor diagrams of various categories from Figure 2.17.

By separating the categories, the graph will look like Figure 2.18, with peak factors calculated for the domestic consumption only, then for the total consumption (excluding leakage), and finally for the total demand (consumption plus leakage). It clearly shows that contributions from the industrial consumption and leakage flatten the patterns.

2.3.3 *Periodic variations*

The peak factors from diurnal diagrams are derived on the basis of average consumption during 24 hours. This average is subject to two additional cycles: weekly and annual.

Weekly demand pattern

Weekly demand pattern is influenced by average consumption on working and non-working days. Public holidays, sport events, etc. play a role in this case as well. One example of the demand variations during a week is shown in Figure 2.19. The difference between the two curves in this diagram reflects the successful implementation of the leak detection programme.

Consumption in urban areas of Western Europe is normally lower over weekends. On Saturdays and Sundays people rest, which may differ in

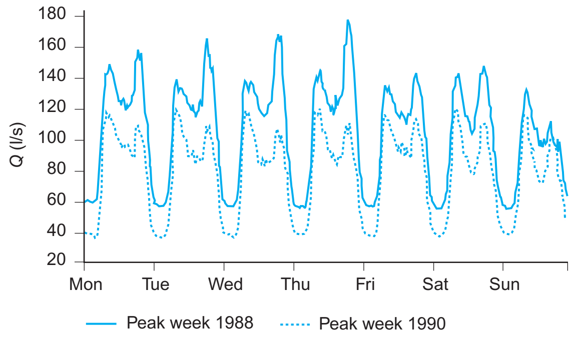


Figure 2.19. Weekly demand variations – Alvington, UK (Dovey and Rogers, 1993).

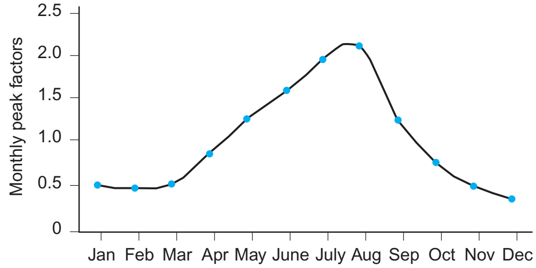


Figure 2.20. Seasonal demand variation in a sea resort (Obradović, 1991).

other parts of the world. For instance, Friday is a non-working day in Islamic countries and domestic consumption usually increases then.

Seasonal variations

Annual variations in water use are predominantly linked to the change of seasons and are therefore also called *seasonal variations*.

The unit consumption per capita normally grows during hot seasons but the increase in total demand may also result from a temporarily increased number of consumers, which is typical for holiday resorts. Figure 2.20 shows the annual pattern in Istria, Croatia on the Adriatic coast; the peaks of the tourist season, during July and August, are also the peaks in water use.

Just as with diurnal patterns, typical weekly and annual patterns can also be expressed through peak factor diagrams. Figure 2.21 shows an example in which the peak daily demand appears typically on Mondays and is 14% above the average, while the minimum on Sundays is 14% below the average daily demand for the week. The second curve shows the difference in demand between summer and winter months, fluctuating within a margin of 10%.

Maximum consumption day

Generalising such trends leads to the conclusion that the absolute peak consumption during one year occurs on a day of the week, and in the month when the consumption is statistically the highest. This day is commonly called the *maximum consumption day*. In the above example, the

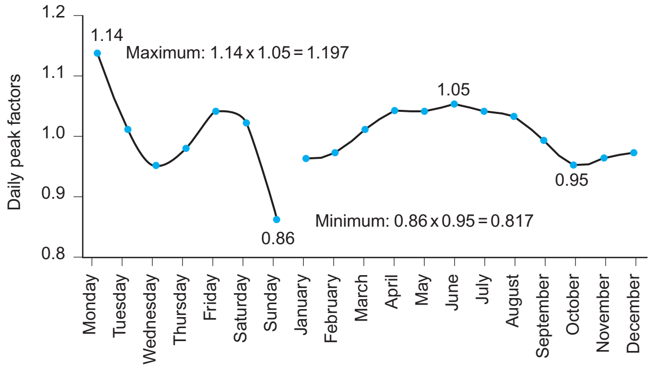


Figure 2.21. Weekly and monthly peak factor diagrams.

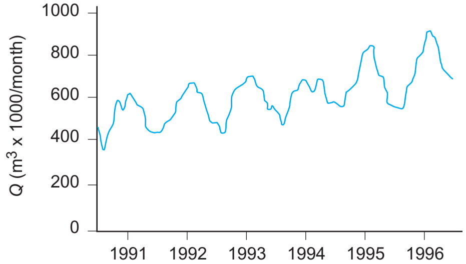


Figure 2.22. Annual demand patterns in Ramallah, Palestine (Thaher, 1998).

maximum consumption day would be a Monday somewhere in June, with its consumption being $1.14 \times 1.05 = 1.197$ times higher than the average daily consumption for the year. In practice however, the maximum consumption day in one distribution area will be determined from the daily demand records of the water company. This is simply the day when the total registered demand was the highest in a particular year.

Finally, the daily, weekly and annual cycles are never repeated in exactly the same way. However, for design purposes a sufficient accuracy is achieved if it is assumed that all water needs are satisfied in a similar schedule during one day, week or year. Regarding the seasonal variations, the example in Figure 2.22 confirms this; the annual patterns in the graph are more or less the same while the average demand grows each year as a result of population growth.

PROBLEM 2.3

A water supply company delivered an annual quantity of 10,000,000 m^3, assuming an average leakage of 20%. On the maximum consumption day, the registered delivery was as follows:

Hour	1	2	3	4	5	6	7	8	9	10	11	12
m^3	989	945	902	727	844	1164	1571	1600	1775	1964	2066	2110
Hour	13	14	15	16	17	18	19	20	21	22	23	24
m^3	1600	1309	1091	945	1062	1455	1745	2139	2110	2037	1746	1018

Determine:

a diurnal peak factors for the area,
b the maximum seasonal variation factor,
c diurnal consumption factors.

Answers:

a From the above table, the average consumption on the maximum consumption day was 1454.75 m^3/h leading to the following hourly peak factors:

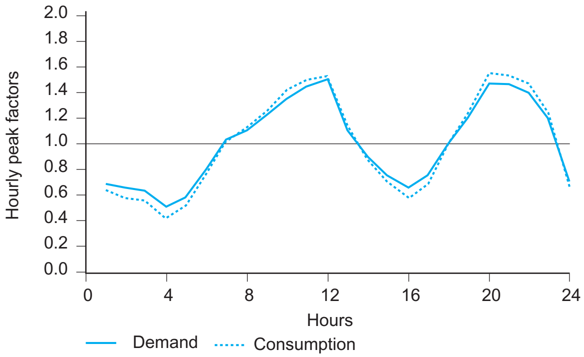
Hour	1	2	3	4	5	6	7	8	9	10	11	12
pf_h	0.680	0.650	0.620	0.500	0.580	0.800	1.080	1.100	1.220	1.350	1.420	1.450
Hour	13	14	15	16	17	18	19	20	21	22	23	24
pf_h	1.100	0.900	0.750	0.650	0.730	1.000	1.200	1.470	1.450	1.400	1.200	0.700

b The average consumption, based on the annual figure, is 10,000,000/365/24 = 1141.55 m^3/h. The seasonal variation factor is therefore 1454.75/1141.55 = 1.274.

c The average leakage of 20% assumes an hourly flow of approx. 228 m^3/h, which is included in the above hourly flows as water loss. The peak factors for consumption will therefore be recalculated without this figure, as the following table shows:

Hour	1	2	3	4	5	6	7	8	9	10	11	12
m^3	761	717	674	499	616	936	1343	1372	1547	1736	1838	1882
pf_h	0.620	0.584	0.549	0.407	0.502	0.763	1.095	1.118	1.261	1.415	1.498	1.534
Hour	13	14	15	16	17	18	19	20	21	22	23	24
m^3	1372	1081	863	717	834	1227	1517	1911	1882	1809	1518	790
pf_h	1.118	0.881	0.703	0.584	0.680	1.000	1.237	1.558	1.534	1.475	1.237	0.644

The diagram of the hourly peak factors for the two situations will look as follows:



Self-study:
Workshop problems A1.1.6 and A1.1.8 (Appendix 1)
Spreadsheet lessons A5.8.2–A5.8.4 (Appendix 5)

2.4 DEMAND CALCULATION

Knowing the daily patterns and periodical variations, the demand flow can be calculated from the following formula:

$$Q_d = \frac{Q_{wc,avg} \times pf_o}{(1 - l/100) \times f_c} \tag{2.1}$$

The definition of the parameters is as follows: Q_d is the water demand of a certain area at a certain moment, $Q_{wc,avg}$ the average water consumption in the area, pf_o the overall peak factor (this is a combination of the peak factor values from the daily, weekly and annual diagrams: $pf_o = pf_h \times pf_d \times pf_m$; the daily and monthly peak factors are normally integrated into one (seasonal) peak factor: $pf_s = pf_d \times pf_m$), l the leakage expressed as a percentage of the water production and f_c the unit conversion factor.

The main advantage of Equation 2.1 is its simplicity although some inaccuracy will be necessarily introduced. Using this formula, the volume of leakage increases with higher consumption i.e. the peak factor value, despite the fixed leakage percentage. For example, if $Q_{wc,avg} = 1$ (regardless of the flow units), $pf_o = 1$ and the leakage percentage is 50%, then as a result $Q_d = 2$. Thus, half of the supply is consumed and the other half is leaked.

If $pf_o = 2$, $Q_d = 4$. Again, this is 'fifty-fifty' but this time the volume of leakage has grown from 1 to 2, which implies its dependence on the consumption level. This is not true as the leakage level is usually constant throughout the day, with a slight increase over night when the pressures in the network are generally higher (already shown in Figure 2.17). Hence, *the leakage level is pressure dependent rather than consumption dependant*.

Nonetheless, the above inaccuracy effectively adds safety to the design. Where this is deemed unnecessary, an alternative approach is suggested, especially for distribution areas with high leakage percentages:

$$Q_d = (Q_{wc,avg} \times pf_o + Q_{wl}) \frac{1}{f_c} \tag{2.2}$$

where:

$$Q_{wl} = \frac{l}{100} \times Q_{wp} \tag{2.3}$$

In the case of $pf_o = 1$, demand equals production and assuming the same units for all parameters ($f_c = 1$):

$$Q_{wp} = Q_{wc,avg} + \frac{l}{100} \times Q_{wp} \tag{2.4}$$

This can be re-written as:

$$Q_{wp} = \frac{Q_{wc,avg}}{(1 - l/100)} \tag{2.5}$$

By plugging Equation 2.5 into 2.3 and then to 2.2, the formula for water demand calculation evolves into its final form:

$$Q_d = \frac{Q_{wc,avg}}{f_c} \times \left(pf_o + \frac{l}{100 - l}\right) \tag{2.6}$$

Where reliable information resulting from individual metering of consumers is not available, the average water consumption, $Q_{wc,avg}$, can be approximated in several ways:

$$Q_{wc,avg} = ncq \tag{2.7}$$

$$Q_{wc,avg} = dAcq \tag{2.8}$$

$$Q_{wc,avg} = Acq_a \tag{2.9}$$

$$Q_{wc,avg} = n_u q_u \tag{2.10}$$

where n is the number of inhabitants in the distribution area, c coverage of the area. It can happen that some of the inhabitants are not connected to the system, or some parts of the area are not inhabited. This factor, which has a value of between 0 and 1, converts the number of inhabitants into the number of consumers. q is the specific consumption (l/c/d), d the population density (number of inhabitants per unit surface area), A the surface area of the distribution area, q_a the consumption registered per unit surface area, n_u the production capacity (it represents a number of units (kg, l, pieces, etc.) produced within a certain period), q_u the water consumption per unit product.

Unit consumptions, q, q_a, and q_u, are elaborated in Paragraph 2.2. The data for n, c, d, A and n_u are usually available from statistics or set by planning: local, urban, regional, etc.

As already mentioned, the demand in large urban areas is often composed of several consumption categories. More accuracy in the calculation of demand for water is therefore achieved if the distribution area is split into a number of sub-areas or districts, with standardised categories of water users and a range of consumptions based on local experience.

The average consumption per district can then be calculated from Equation 2.9, which has been modified:

$$Q_{\mathrm{wc,avg}} = A\sum_{i=1}^{n}(q_{\mathrm{a},i}p_i c_i) \tag{2.11}$$

where A is the surface area of the district, n the number of consumption categories within the district, $q_{\mathrm{a},i}$ the unit consumption per surface area of category i, p_i the percentage of the district territory occupied by category i, c_i the coverage within the district territory occupied by category i. With a known population density in each district, the result can be converted into specific demand (per capita).

Regarding the pf_o values, the following are typical combinations:

1 $pf_\mathrm{h} = 1$, $pf_\mathrm{s} = 1$; Q_d represents the average consumption per day. This demand is the absolute average, usually obtained from annual demand records and converted into required flow units.
2 $pf_\mathrm{h} = 1$, $pf_\mathrm{s} = \max$; Q_d represents the average demand during the maximum consumption day.
3 $pf_\mathrm{h} = \max$, $pf_\mathrm{s} = \max$; Q_d is the demand at the maximum consumption hour on the maximum consumption day.
4 $pf_\mathrm{h} = \min$, $pf_\mathrm{s} = \min$; Q_d is the demand at the minimum consumption hour on the minimum consumption day.

The entire range of demands that appear in one distribution system during one year is specified by the demands under 3 and 4, which are shown in Figure 2.23. These peak demands are relevant as parameters for the design of all system components: pipes, pumps and storage.

PROBLEM 2.4
A water supply company in a town with a total population of approx. 275,000 conducted a water demand survey resulting in the following

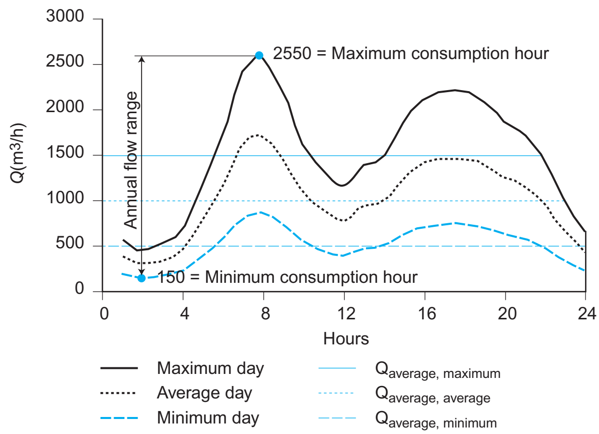


Figure 2.23. Hypothetical annual range of flows in a distribution system.

categories of water users:

Category of water users	q_a (m^3/d/ha)
A Residential area, apartments	90
B Residential area, individual houses	55
C Shopping areas	125
D Offices	80
E Schools, Colleges	100
F Hospitals	160
G Hotels	150
H Public green areas	15

The city is divided into 8 districts, each with a known population, contribution to demand from each of the categories, and estimated coverage by the distribution system, as shown in the table below.

Districts		A	B	C	D	E	F	G	H
86,251	p_1(%)	37	23	10	0	4	0	0	26
A_1=250 ha	c_1(%)	100	100	100	0	100	0	0	40
74,261	p_2(%)	20	5	28	11	12	0	5	19
A_2=185 ha	c_2(%)	100	100	95	100	100	0	100	80
18,542	p_3(%)	10	18	3	0	0	42	0	27
A_3=57 ha	c_3(%)	100	100	100	0	0	100	0	35
42,149	p_4(%)	25	28	20	2	15	0	0	10
A_4=88 ha	c_4(%)	100	100	95	100	100	0	0	36
22,156	p_5(%)	50	0	11	0	10	0	0	29
A_5=54 ha	c_5(%)	100	0	100	0	100	0	0	65
9958	p_6(%)	24	11	13	15	13	8	0	16
A_6=29 ha	c_6(%)	100	100	100	100	100	100	0	35
8517	p_7(%)	22	28	8	19	6	0	10	7
A_7=17 ha	c_7(%)	100	100	100	100	100	0	100	50
12,560	p_8(%)	0	0	0	0	55	20	15	10
A_8=16 ha	c_8(%)	0	0	0	0	85	100	100	45

Determine the total average demand of the city.

Answer:
Based on Equation 2.11, a sample calculation of the demand for district 1 will be as follows:

$$
\begin{aligned}
Q_{1,\text{avg}} &= A\sum_{i=1}^{5}(q_{a,i}p_i c_i) \\
&= 250 \times (90 \times 0.37 + 55 \times 0.23 + 125 \times 0.1 \\
&\quad + 100 \times 0.04 + 15 \times 0.26 \times 0.40) \\
&= 666.77\ \text{m}^3/\text{h}
\end{aligned}
$$

The remainder of the results are shown in the table below. The specific demand has been calculated based on the registered population in each district.

	Q_{avg}(m^3/h)	Population	Q_{avg}(l/c/d)
District 1	666.67	86,251	186
District 2	651.97	74,261	211
District 3	216.76	18,542	281
District 4	288.90	42,149	165
District 5	161.05	22,156	174
District 6	99.74	9958	240
District 7	58.03	8517	164
District 8	67.95	12,560	130
Total	2211.16	274,394	193

Self-study:
Spreadsheet lesson A5.8.5–A5.8.7 (Appendix 5)

2.5 DEMAND FORECASTING

Water demand usually grows unpredictably as it depends on many parameters that have their own unpredictable trends. Figure 2.24 illustrates how the rate of increase in consumption may differ even in countries from the same region and with a similar level of economic development.

The experience from Germany proves again how uncertain the forecast can be. Figure 2.25 shows the development of domestic consumption in the period 1970–2000. The forecast made by experts in the Seventies and the Eighties was that the demand in the year 2000 would

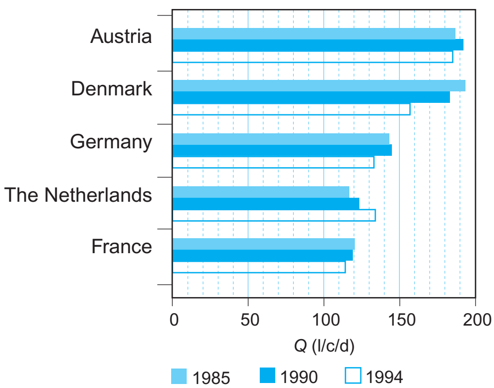


Figure 2.24. Domestic consumption increase in some European countries (EEA, 2001).

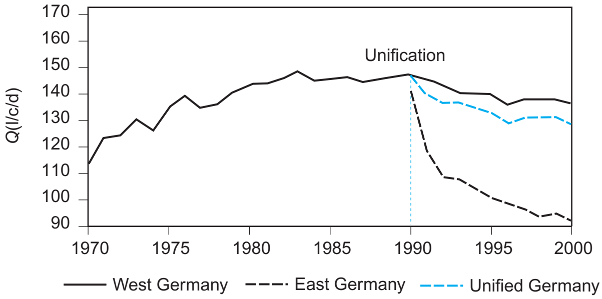


Figure 2.25. Domestic consumption increase in Germany (BGW).

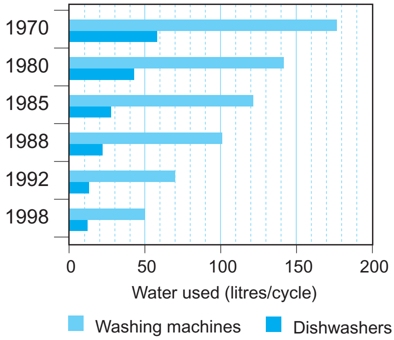


Figure 2.26. Water consumption of washing appliances in Europe (EEA, 2001).

grow to as much as 220 l/c/d, while in reality it fell to approximately 140 l/c/d.

Awareness for the environment in the last decade, combined with low population growth, caused a drop in domestic water use in many countries of Western Europe. In addition, lots of home appliances (i.e. shower heads, taps, washing machines, dishwashers, etc.) have been replaced with more advanced models, able to achieve the same effect with less water (see Figure 2.26). Finally, industry has been moving towards alternative water-saving production technologies, positively influencing the overall water demand. This is unfortunately not the case in many developing countries, where the population growth is much higher, consumers' attitude towards water conservation is comparatively low, and outdated technologies and equipment are still widely used.

Apart from monitoring technological developments, several other assessments must be taken into account while estimating future demand:

– historical demand growth trends,
– projection-based on per capita consumption and population growth trends for domestic category,
– forecast-based on assessment of growth trends of other main consumer categories (industry, tourism, etc.).

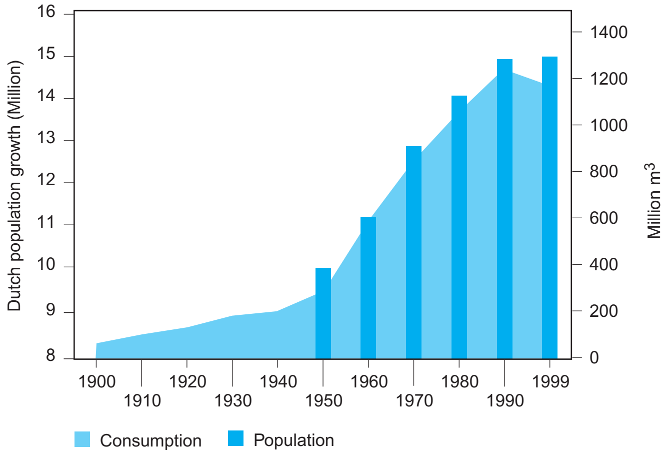


Figure 2.27. Population and demand growth in The Netherlands (VROM).

When combined, all these projections can yield several possible scenarios of consumption growth. While thinking, for instance, about population growth, which is for many developing countries still the major factor in an increase in water demand, useful conclusions can be drawn if the composition of the existing population, fertility and mortality rates, and particularly the rate of migration, can be assessed. That the population and demand growth match reasonably well in general is shown by the example in Figure 2.27.

Two models are commonly used for demand forecast: linear and exponential.
Linear model

$$Q_{i+n} = Q_i\left(1 + n\frac{a}{100}\right) \tag{2.12}$$

Exponential model

$$Q_{i+n} = Q_i\left(1 + \frac{a}{100}\right)^n \tag{2.13}$$

In the above equations Q_i is the water demand at year i, Q_{i+n} the forecast water demand after n years, n the design period, a the average annual population growth during the design period (%).

Which of the models will be more suitable will depend on the conclusions from the above-mentioned analyses. These are to be reviewed periodically, as trends can change within a matter of years.

Figure 2.28 shows the annual demand in The Netherlands in the period 1955–1995. In the first part of this period, up until 1970, the

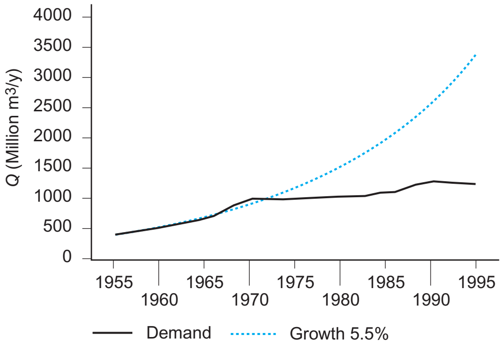


Figure 2.28. Consumption growth in The Netherlands according to the exponential model.

exponential model with an average annual growth of 5.5% (a in Equation 2.13 = 5.5) matches the real demand very closely. However, keeping it unchanged for the entire period would show demand almost three times higher than was actually registered in 1995.

PROBLEM 2.5
In a residential area of 250,000 inhabitants, the specific water demand is estimated at 150 l/c/d, which includes leakage. Calculate the demand in 20 years' time if the assumed annual demand growth is 2.5%. Compare the results by applying the linear and exponential models.

Answers:
The present demand in the city is equal to:

$$Q_{avg} = \frac{250{,}000 \times 150 \times 365}{1000} = 13{,}687{,}500 \text{ m}^3/\text{y}$$

Applying the linear model, the demand after 20 years will grow to:

$$Q_{21} = 13{,}687{,}500\left(1 + 20\frac{2.5}{100}\right) = 20{,}531{,}250 \text{ m}^3/\text{y}$$

which is an increase of 50% compared to the present demand. In the case of the exponential model:

$$Q_{21} = 13{,}687{,}500\left(1 + \frac{2.5}{100}\right)^{20} \approx 22{,}428{,}600 \text{ m}^3/\text{y}$$

which is an increase of approximately 64% compared to the present demand.

Self-study:
Workshop problems A1.1.9 and A1.1.10 (Appendix 1)
Spreadsheet lesson A5.8.8 (Appendix 5)

2.6 DEMAND FREQUENCY DISTRIBUTION

A water supply system is generally designed to provide the demand at guaranteed minimum pressures, for 24 hours a day and 365 days per year. Nevertheless, if the pressure threshold is set high, such a level of service may require exorbitant investment that is actually non-affordable for the water company and consumers. It is therefore useful to analyse how often the maximum peak demands occur during the year. The following example explains the principle.

Knowing both a typical diurnal peak factor diagram (such as the one shown in Figure 2.29) and the range of seasonal peak factors allows for integration of the two. The hourly peak factors corrected by the seasonal peak factors will result in the annual range of the hourly peak hours (Figure 2.30). These are absolute values that refer to the average hour of the average consumption day (Figure 2.31). Consequently, each hour of the year (total 24 × 365) will have a unique peak factor value assigned to it.

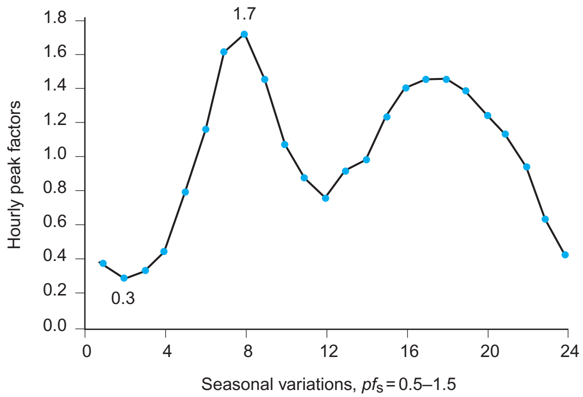


Figure 2.29. Example of a typical diurnal demand pattern.

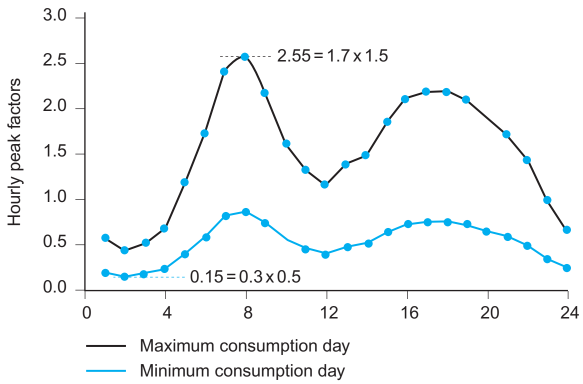


Figure 2.30. Example of the annual range of the peak factors.

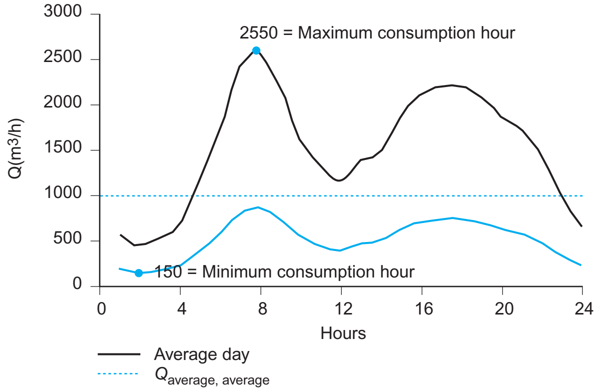


Figure 2.31. Example of the annual range of hourly demands.

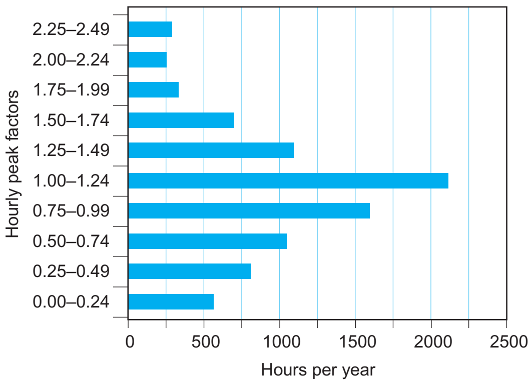


Figure 2.32. Frequency distribution of the diurnal peak factors.

Applying this logic, the diagram with the frequency distribution of all hourly peak factors can be plotted, as the example in Figure 2.32 shows. Converting this diagram into a cumulative frequency distribution curve helps to determine the number of hours in the year when the peak factors exceed the corresponding value. This is illustrated in Figure 2.33, which for instance shows that the peak factors above 2.0 only appear during some 500 hours or approximately 5% of the year. In theory, excluding this fraction from the design considerations would eventually create savings based on a 20% reduction of the system capacity. The consequence of such a choice would be the occasional drop of pressure below the threshold, which the consumers might consider acceptable during a limited period of time.

In practice, the decision about the design peak factor results from the comparison between the costs and benefits. Indeed, it seems rather inefficient to lay pipes that will be used for 80% of their capacity for less than 5% of the total time. However, where there is a considerable scope for energy savings from daily use by lowering the energy losses on a

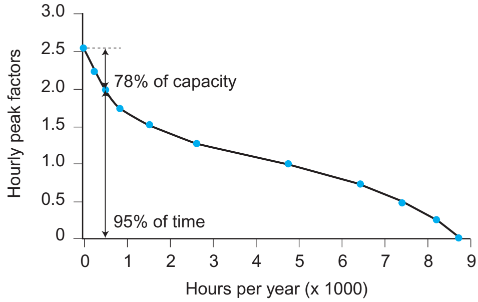


Figure 2.33. Cumulative frequency distribution of the diurnal peak factors.

wider scale, such a choice may look reasonable. Moreover, careful assessment of the network reliability could justify the laying of pipes with reserve capacity that could be utilised during irregular supply situations. Finally, some spare capacity is also useful for practical reasons since it can postpone the construction of phased extensions to expand the system.

The relation between demand for water and hydraulic losses is thoroughly discussed in the following chapter.

Self-study:
Spreadsheet lesson A5.8.9 (Appendix 5).

CHAPTER 3

Steady Flows in Pressurised Networks

3.1 MAIN CONCEPTS AND DEFINITIONS

The basic hydraulic principles applied in water transport and distribution practice emerge from three main assumptions:
1 The system is filled with water under pressure,
2 that water is incompressible,
3 that water has a steady and uniform flow.

In addition, it is assumed that the deformation of the system boundaries is negligible, meaning that the water flows through a non-elastic system.[1]

Steady flow

Flow Q (m^3/s) through a pipe cross-section of area A (m^2) is determined as $Q = \upsilon \times A$, where υ (m/s) is the mean velocity in the cross-section. This flow is *steady* if the mean velocity remains constant over a period of time Δt.

Uniform flow

If the mean velocities of two consecutive cross-sections are equal at a particular moment, the flow is *uniform*.

The earlier definitions written in the form of equations for two close moments, t_1 and t_2, and in the pipe cross-sections 1 and 2 (Figure 3.1) yield:

$$v_1^{(t_1)} = v_1^{(t_2)} \wedge v_2^{(t_1)} = v_2^{(t_2)} \tag{3.1}$$

for a steady flow, and:

$$v_1^{(t_1)} = v_2^{(t_1)} \wedge v_1^{(t_2)} = v_2^{(t_2)} \tag{3.2}$$

for a uniform flow.

A steady flow in a pipe with a constant diameter is at the same time uniform. Thus:

$$v_1^{(t_1)} = v_2^{(t_1)} = v_1^{(t_2)} = v_2^{(t_2)} \tag{3.3}$$

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[1] The foundations of steady state hydraulics are described in detail in various references of Fluid Mechanics and Engineering Hydraulics. See for instance Streeter and Wylie (1985).

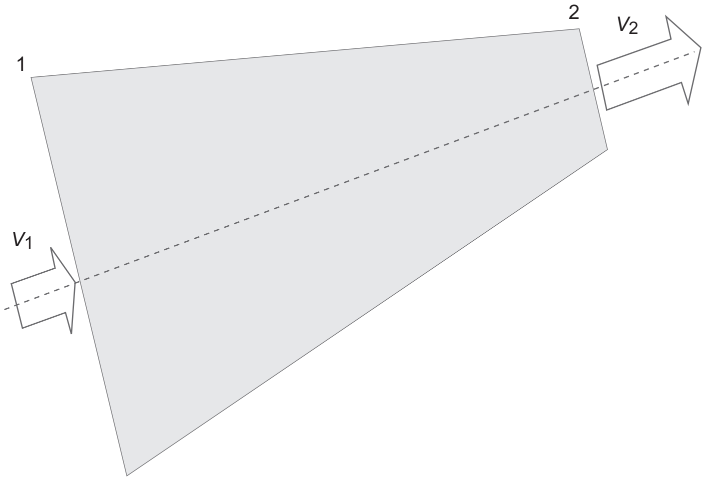


Figure 3.1. Steady and uniform flow.

Transient flow

The earlier simplifications help to describe the general hydraulic behaviour of water distribution systems assuming that the time interval between t_1 and t_2 is sufficiently short. Relatively slow changes of boundary conditions during regular operation of these systems make Δt of a few minutes acceptably short for the assumptions introduced earlier. At the same time, this interval is long enough to simulate changes in pump operation, levels in reservoirs, diurnal demand patterns, etc., without handling unnecessarily large amounts of data. If there is a sudden change in operation, for instance a situation caused by pump failure or valve closure, transitional flow conditions occur in which the assumptions of the steady and uniform flow are no longer valid. To be able to describe these phenomena in a mathematically accurate way, a more complex approach elaborated in the theory of *transient flows* would be required, which is not discussed in this book. The reference literature on this topic includes Larock *et al*. (2000).

3.1.1 *Conservation laws*

The conservation laws of mass, energy and momentum are three fundamental laws related to fluid flow. These laws state:

1 *The Mass Conservation Law*
  Mass m (kg) can neither be created nor destroyed; any mass that enters a system must either accumulate in that system or leave it.
2 *The Energy Conservation Law*
  Energy E (J) can neither be created nor destroyed; it can only be transformed into another form.

3 *The Momentum Conservation Law*
The sum of external forces acting on a fluid system equals the change of the momentum rate M (N) of that system.

The conservation laws are translated into practice through the application of three equations, respectively:
1 The Continuity Equation.
2 The Energy Equation.
3 The Momentum Equation.

Continuity Equation

The *Continuity Equation* is used when balancing the volumes and flows in distribution networks. Assuming that water is an incompressible fluid, i.e. with a mass density $\rho = m/V = \text{const}$, the Mass Conservation Law can be applied to volumes. In this situation, the following is valid for tanks (see Figure 3.2):

$$Q_{inp} = Q_{out} \pm \frac{\Delta V}{\Delta t} \tag{3.4}$$

where $\Delta V/\Delta t$ represents the change in volume V (m^3) within a time interval Δt (s). Thus, the difference between the input- and output-flow from a tank is the volume that is:
1 accumulated in the tank if $Q_{out} < Q_{inp}$ (sign + in Equation 3.4),
2 withdrawn from the tank if $Q_{out} > Q_{inp}$ (sign −).
Applied at node n that connects j pipes, the Continuity Equation can be written as:

$$\sum_{i=1}^{j} Q_i - Q_n = 0 \tag{3.5}$$

where Q_n represents the nodal discharge. An example of three pipes and a discharge point is shown in Figure 3.3.

Energy Equation

The *Energy Equation* establishes the energy balance between any two cross-sections of a pipe:

$$E_1 = E_2 \pm \Delta E \tag{3.6}$$

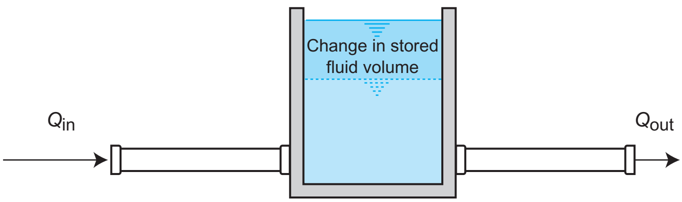


Figure 3.2. The Continuity Equation validity in tanks.

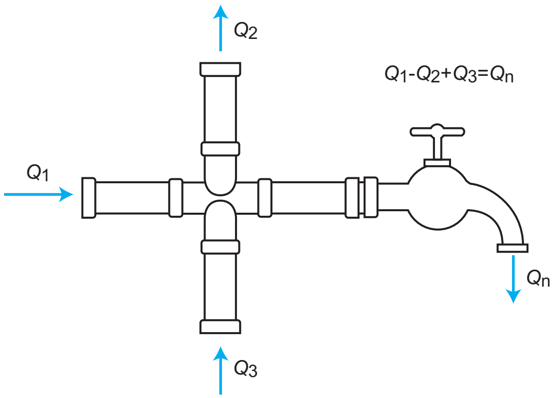


Figure 3.3. The Continuity Equation validity in pipe junctions.

where ΔE is the amount of transformed energy between cross-sections 1 and 2. It is usually the energy lost from the system (the sign + in Equation 3.6), but may also be added to it by pumping of water (sign −).

Momentum Equation

The *Momentum Equation* (in some literature also known as the Dynamic Equation) describes the pipe resistance to dynamic forces caused by the pressurised flow. For incompressible fluids, momentum M (N) carried across a pipe section is defined as:

$$M = \rho Q v \tag{3.7}$$

where ρ (kg/m^3) represents the mass density of water, Q(m^3/s) is the pipe flow, v (m/s) is the mean velocity. Other forces in the equilibrium are (see Figure 3.4):

1 Hydrostatic force F_h (N) caused by fluid pressure p (N/m^2 or Pa); $F_h = p \times A$.
2 Weight w (N) of the considered fluid volume (only acts in a vertical direction).
3 Force F (N) of the solid surface acting on the fluid.

The Momentum Equation as written for a horizontal direction would state:

$$\rho Q v_1 - \rho Q v_2 \cos\varphi = -p_1 A_1 + p_2 A_2 \cos\varphi + F_x \tag{3.8}$$

whereas in a vertical direction:

$$\rho Q v_2 \sin\varphi = -p_2 A_2 \sin\varphi + w + F_y \tag{3.9}$$

Pipe thrust

The forces of the water acting on the pipe bend are the same, i.e. F_x and F_y but with an opposite direction i.e. a negative sign, in which case the

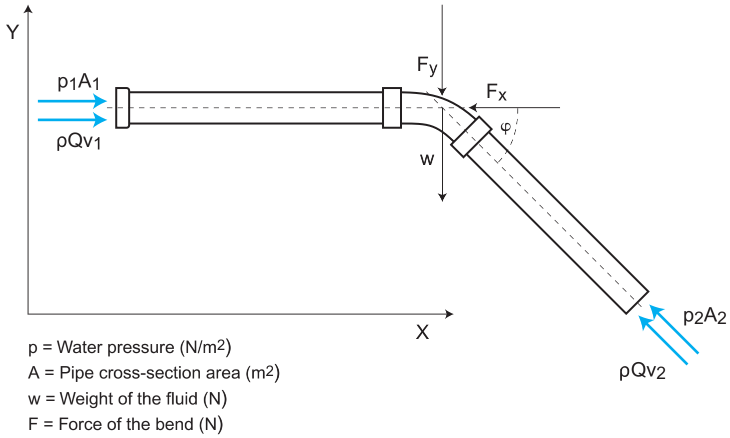


Figure 3.4. The Momentum Equation.

total force, known as the *pipe thrust* will be:

$$F = \sqrt{F_x^2 + F_y^2} \tag{3.10}$$

The Momentum Equation is applied in calculations for the additional strengthening of pipes, in locations where the flow needs to be diverted. The results are used for the design of concrete structures required for anchoring of pipe bends and elbows.

PROBLEM 3.1
A velocity of 1.2 m/s has been measured in a pipe of diameter $D = 600$ mm. Calculate the pipe flow.

Answer:
The cross-section of the pipe is:

$$A = \frac{D^2\pi}{4} = \frac{0.6^2 \times 3.14}{4} = 0.2827 \text{ m}^2$$

which yields the flow of:

$$D = vA = 1.2 \times 0.2827 = 0.339 \text{ m}^3/\text{s} \approx 340 \text{ l/s}$$

PROBLEM 3.2
A circular tank with a diameter at the bottom of $D = 20$ m and with vertical walls has been filled with a flow of 240 m^3/h. What will be the increase of the tank depth after 15 minutes, assuming a constant flow during this period of time?

Answer:
The tank cross-section area is:

$$A = \frac{D^2\pi}{4} = \frac{20^2 \times 3.14}{4} = 314.16 \text{ m}^2$$

The flow of 240 m^3/h fills the tank with an additional 60 m^3 after 15 minutes, which is going to increase the tank depth by a further 60/314.16 = 0.19 m ≈ 20 cm.

PROBLEM 3.3
For a pipe bend of 45° and a continuous diameter of $D = 300$ mm, calculate the pipe thrust if the water pressure in the bend is 100 kPa at a measured flow rate of 26 l/s. The weight of the fluid can be neglected. The mass density of the water equals $\rho = 1000$ kg/m^3.

Answer:
From Figure 3.4, for a continuous pipe diameter:

$$A_1 = A_2 = \frac{D^2\pi}{4} = \frac{0.3^2\times 3.14}{4} = 0.07\,\mathrm{m}^2$$

Consequently, the flow velocity in the bend can be calculated as:

$$v_1 = v_2 = \frac{Q}{A} = \frac{0.026}{0.07} = 0.37\,\mathrm{m/s}$$

Furthermore, for the angle $\varphi = 45°$, $\sin\varphi = \cos\varphi = 0.71$. Assuming also that $p_1 = p_2 = 100$ kPa (or 100,000 N/m^2), the thrust force in the X-direction becomes:

$$\begin{aligned} -F_x &= 0.29\times(pA + \rho Qv) = 0.29\times(100{,}000\times 0.07 \\ &\quad + 1000\times 0.026\times 0.37) \approx 2030\,\mathrm{N} = 2\,\mathrm{kN} \end{aligned}$$

while in the Y-direction:

$$-F_y = 0.71\times(pA + \rho Qv) \approx 5\,\mathrm{kN}$$

The total force will therefore be:

$$F = \sqrt{2^2 + 5^2} \approx 5.4\,\mathrm{kN}$$

The calculation shows that the impact of water pressure is much more significant that the one of the flow/velocity.

Self-study:
Spreadsheet lesson A5.1.1 (Appendix 5)

3.1.2 *Energy and hydraulic grade lines*

The energy balance in Equation 3.6 stands for total energies in two cross-sections of a pipe. The total energy in each cross-section comprises three components, which is generally written as:

$$E_{\mathrm{tot}} = mgZ + m\frac{p}{\rho} + \frac{mv^2}{2} \qquad (3.11)$$

expressed in J or more commonly in kWh. Written per unit weight, the equation looks as follows:

$$E_{tot} = Z + \frac{p}{\rho g} + \frac{v^2}{2g} \tag{3.12}$$

where the energy obtained will be expressed in *metres water column* (mwc). Parameter g in both these equations stands for gravity (9.81 m/s^2).

Potential energy

The first term in Equations 3.11 and 3.12 determines the *potential energy*, which is entirely dependant on the elevation of the mass/volume.

The second term stands for the flow energy that comes from the ability of a fluid mass $m = \rho \times V$ to do work W (N) generated by the earlier-mentioned pressure forces $F = p \times A$. At pipe length L, these forces create the work that can be described per unit mass as:

$$W = FL = \frac{pAL}{\rho V} = \frac{p}{\rho} \tag{3.13}$$

Kinetic energy

Finally, the third term in the equations represents the *kinetic energy* generated by the mass/volume motion.

By plugging 3.12 into 3.6, it becomes:

$$Z_1 + \frac{p_1}{\rho g} + \frac{v_1^2}{2g} = Z_2 + \frac{p_2}{\rho g} + \frac{v_2^2}{2g} \pm \Delta E \tag{3.14}$$

Bernoulli Equation

In this form, the energy equation is known as the *Bernoulli Equation*. The equation parameters are shown in Figure 3.5. The following terminology is in common use:

– *Elevation head*: $Z_{1(2)}$
– *Pressure head*: $p_{1(2)}/\rho g$
– *Piezometric head*: $H_{1(2)} = Z_{1(2)} + p_{1(2)}/\rho g$
– *Velocity head*: $v^2{}_{1(2)}/2g$
– *Energy head*: $E_{1(2)} = H_{1(2)} + v^2{}_{1(2)}/2g$

The pressure- and velocity-heads are expressed in mwc, which gives a good visual impression while talking about ‘high-’ or ‘low’ pressures or energies. The elevation-, piezometric- and energy heads are compared to a reference or ‘zero’ level. Any level can be taken as a reference; it is commonly the mean sea level suggesting the units for Z, H and E in *metres above mean sea level* (msl). Alternatively, the street level can also be taken as a reference level.

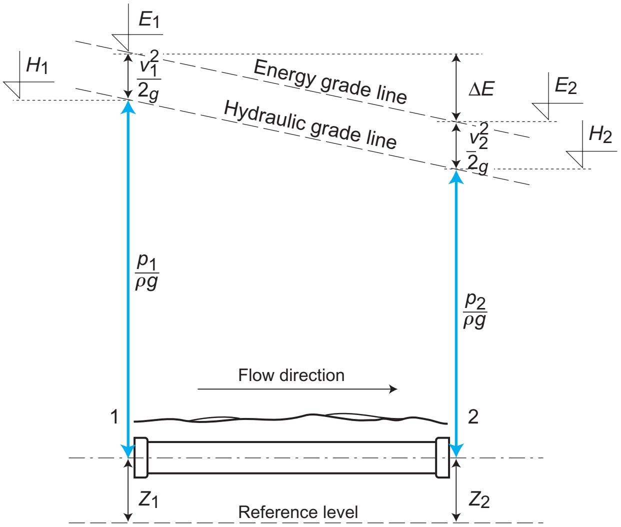


Figure 3.5. The Bernoulli Equation.

To provide a link with the SI-units, the following is valid:

- 1 mwc of the pressure head corresponds to 9.81 kPa in SI-units, which for practical reasons is often rounded off to 10 kPa.
- 1 mwc of the potential energy corresponds to 9.81 ($\approx$10) kJ in SI-units; for instance, this energy will be possessed by 1 m^3 of the water volume elevated 1 m above the reference level.
- 1 mwc of the kinetic energy corresponds to 9.81 ($\approx$10) kJ in SI-units; for instance, this energy will be possessed by 1 m^3 of the water volume flowing at a velocity of 1 m/s.

In reservoirs with a surface level in contact with the atmosphere, pressure p equals the atmospheric pressure, hence $p = p_{atm} \approx 0$. Furthermore, the velocity throughout the reservoir volume can be neglected ($\upsilon \approx 0$ m/s). As a result, both the energy- and piezometric-head will be positioned at the surface of the water. Hence, $E_{tot} = H = Z$.

Energy and Hydraulic grade line

The lines that indicate the energy- and piezometric-head levels in consecutive cross-sections of a pipe are called the *energy grade line* and the *hydraulic grade line*, respectively.

The energy and hydraulic grade line are parallel for uniform flow conditions. Furthermore, the velocity head is in reality considerably smaller than the pressure head. For example, for a common pipe velocity of 1 m/s, $v^2/2g = 0.05$ mwc, while the pressure heads are often in the order of tens of metres of water column. Hence, the real difference between these two lines is, with a few exceptions, negligible and the hydraulic grade line is predominantly considered while solving practical

problems. Its position and slope indicate:
– the pressures existing in the pipe, and
– the flow direction.

The hydraulic grade line is generally not parallel to the slope of the pipe that normally varies from section to section. In hilly terrains, the energy level may even drop below the pipe invert causing negative pressure (below atmospheric), as Figure 3.6 shows.

Hydraulic gradient

The slope of the hydraulic grade line is called the *hydraulic gradient*, $S = \Delta E/L = \Delta H/L$, where L (m) is the length of the pipe section. This parameter reflects the pipe conveyance (Figure 3.7).

The flow rate in pipes under pressure is related to the hydraulic gradient and not to the slope of the pipe. More energy is needed for a pipe to convey more water, which is expressed in the higher value of the hydraulic gradient.

PROBLEM 3.4
For the pipe bend in Problem 3.3 (Section 3.1.1), calculate the total energy- and piezometric head in the cross-section of the bend if it is located at $Z = 158$ msl. Express the result in msl, J and kWh.

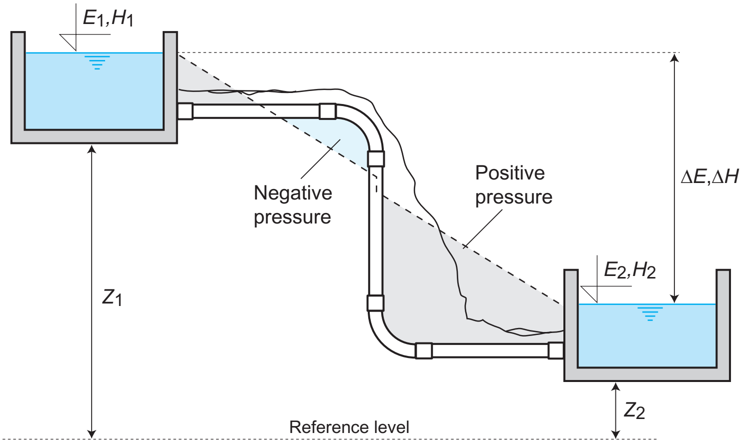


Figure 3.6. Hydraulic grade line.

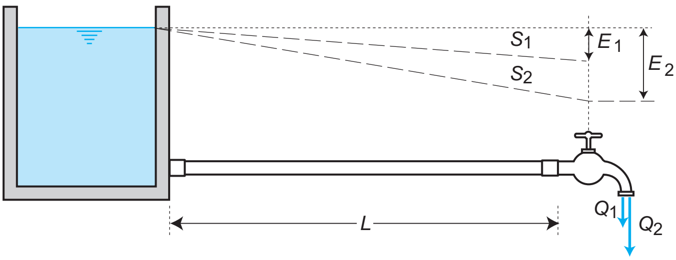


Figure 3.7. The hydraulic gradient.

Answer:
In Problem 3.3, the pressure indicated in the pipe bend was $p = 100$ kPa, while the velocity, calculated from the flow rate and the pipe diameter, was $v = 0.37$ m/s. The total energy can be determined from Equation 3.12:

$$E_{tot} = Z + \frac{p}{\rho g} + \frac{v^2}{2g} = 158 + \frac{100{,}000}{1000 \times 9.81} + \frac{0.37^2}{2 \times 9.81}$$
$$= 158 + 10.194 + 0.007 = 168.2 \text{ msl}$$

As can be seen, the impact of the kinetic energy is minimal and the difference between the total energy and the piezometric head can therefore be neglected. The same result in J and kWh is as follows:

$$E_{tot} = 168.2 \times 1000 \times 9.81 = 1{,}650{,}042 \text{ J} \approx 1650 \text{ kJ } (\textit{or} \text{ kWs})$$
$$= \frac{1650}{3600} \approx 0.5 \text{ kWh}$$

For an unspecified volume, the above result represents a type of unit energy, expressed per m^3 of water. To remember the units conversion: 1 N = 1 kg × m/s^2 and 1 J = 1 N × m.

3.2 HYDRAULIC LOSSES

The energy loss ΔE from Equation 3.14 is generated by:
– friction between the water and the pipe wall,
– turbulence caused by obstructions of the flow.

These causes inflict the *friction*- and *minor losses*, respectively. Both can be expressed in the same format:

$$\Delta E = h_f + h_m = R_f Q^{n_f} + R_m Q^{n_m} \tag{3.15}$$

Pipe resistance

where R_f stands for *resistance* of a pipe with diameter D, along its length L. The parameter R_m can be characterised as a resistance at the pipe cross-section where obstruction occurs. Exponents n_f and n_m depend on the type of equation applied.

3.2.1 *Friction losses*

The most popular equations used for the determination of friction losses are:
1 the Darcy–Weisbach Equation,
2 the Hazen–Williams Equation,
3 the Manning Equation.

Following the format in Equation 3.15:
Darcy–Weisbach

$$R_f = \frac{8\lambda L}{\pi^2 g D^5} = \frac{\lambda L}{12.1 D^5}; \quad n_f = 2 \tag{3.16}$$

Hazen–Williams

$$R_f = \frac{10.68 L}{C_{hw}^{1.852} D^{4.87}}; \quad n_f = 1.852 \tag{3.17}$$

Manning

$$R_f = \frac{10.29 N^2 L}{D^{16/3}}; \quad n_f = 2 \tag{3.18}$$

In all three cases, the friction loss h_f will be calculated in mwc for the flow Q expressed in m^3/s and length L and diameter D expressed in m. *The use of prescribed parameter units in Equations 3.16–3.18 is to be strictly obeyed as the constants will need to be readjusted depending on the alternative units used*.

In the above equations, λ, C_{hw} and N are experimentally-determined factors that describe the impact of the pipe wall roughness on the friction loss.

The Darcy–Weisbach Equation

Colebrook–White Equation

In the Darcy–Weisbach Equation, the friction factor λ $(-)$ (also labelled as f in some literature) can be calculated from the equation of *Colebrook–White*:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{2.51}{Re\sqrt{\lambda}} + \frac{k}{3.7D}\right] \tag{3.19}$$

where k is the absolute roughness of the pipe wall (mm), D the inner diameter of the pipe (mm) and Re the Reynolds number $(-)$.

To avoid iterative calculation, *Barr* (1975) suggests the following acceptable approximation, which deviates from the results obtained by the Colebrook–White Equation for $\pm$ 1%:

$$\frac{1}{\sqrt{\lambda}} = -2\log\left[\frac{5.1286}{Re^{0.89}} + \frac{k}{3.7D}\right] \tag{3.20}$$

Reynolds number

The *Reynolds number* describes the flow regime. It can be calculated as:

$$Re = \frac{vD}{\upsilon} \tag{3.21}$$

Kinematic viscosity

where υ (m^2/s) stands for the *kinematic viscosity*. This parameter depends on the water temperature and can be determined from the following equation:

$$\upsilon = \frac{497 \times 10^{-6}}{(T + 42.5)^{1.5}} \tag{3.22}$$

for T expressed in °C.

The flow is:

1 laminar, if $Re < 2000$,
2 critical (in transition), for $Re \approx 2000$–4000,
3 turbulent, if $Re > 4000$.

The turbulent flows are predominant in distribution networks under normal operation. For example, within a typical range for the following parameters: $v = 0.5$–1.5 m/s, $D = 50$–1500 mm and $T = 10$–20°C, the Reynolds number calculated by using Equations 3.21 and 3.22 has a value of between 19,000 and 225,0000.

If for any reason Re<4000, Equations 3.19 and 3.20 are no longer valid. The friction factor for the laminar flow conditions is then calculated as:

$$\lambda = \frac{64}{Re} \tag{3.23}$$

As it usually results from very low velocities, this flow regime is not favourable in any way.

Once Re, k and D are known, the λ-factor can also be determined from the Moody diagram, shown in Figure 3.8. This diagram is in essence a graphic presentation of the Colebrook–White Equation.

In the turbulent flow regime, Moody diagram shows a family of curves for different k/D ratios. This zone is split in two by the dashed line.

Transitional turbulence zone

The first sub-zone is called the *transitional turbulence zone*, where the effect of the pipe roughness on the friction factor is limited compared to the impact of the Reynolds number (i.e. the viscosity).

Rough turbulence zone

The curves in the second sub-zone of the *rough (developed) turbulence* are nearly parallel, which clearly indicates the opposite situation where the Reynolds number has little influence on the friction factor. As a result, in this zone the Colebrook–White Equation can be simplified:

$$\frac{1}{\sqrt{\lambda}} = -2 \log\left[\frac{k}{3.7D}\right] \tag{3.24}$$

For typical values of v, k, D and T, the flow rate in distribution pipes often drops within the rough turbulence zone.

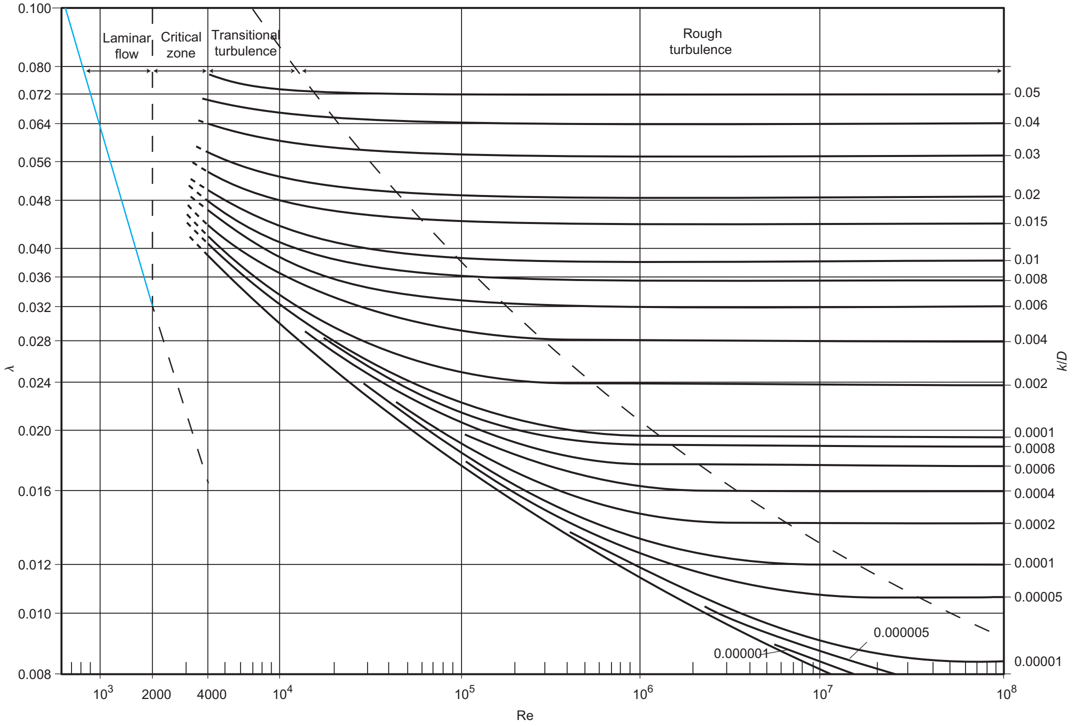


Figure 3.8. Moody diagram.

Table 3.1. Absolute roughness (Wessex Water PLC, 1993).

Pipe material	*k* (mm)
Asbestos cement	0.015–0.03
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, Copper	0.003

Absolute roughness

The *absolute roughness* is dependant upon the pipe material and age. The most commonly used values for pipes in good condition are given in Table 3.1.

With the impact of corrosion, the *k*-values can increase substantially. In extreme cases, severe corrosion will be taken into consideration by reducing the inner diameter.

The Hazen–Williams Equation

The Hazen–Williams Equation is an empirical equation widely used in practice. It is especially applicable for smooth pipes of medium and large diameters and pipes that are not attacked by corrosion (Bhave, 1991). The values of the Hazen–Williams constant, C_{hw} (−), for selected pipe materials and diameters are shown in Table 3.2.

Bhave states that the values in Table 3.2 are experimentally determined for flow velocity of 0.9 m/s. A correction for the percentage given in Table 3.3 is therefore suggested in case the actual velocity differs significantly. For example, the value of $C_{hw} = 120$ increases twice for 3% if the expected velocity is around a quarter of the reference value i.e. $C_{hw} = 127$ for v of, say, 0.22 m/s. On the other hand, for doubled velocity $v = 1.8$ m/s, $C_{hw} = 116$ i.e. 3% less than the original value of 120. However, such corrections do not significantly influence the friction loss calculation, and are, except for extreme cases, rarely applied in practice.

Bhave also states that the Hazen–Williams Equation becomes less accurate for C_{hw}-values below 100.

The Manning Equation

Strickler Equation

The Manning Equation is another empirical equation used for the calculation of friction losses. In a slightly modified format, it also occurs in some literature under the name of *Strickler*. The usual range of the N-values ($m^{-1/3}s$) for typical pipe materials is given in Table 3.4.

Table 3.2. The Hazen–Williams factors (Bhave, 1991).

Pipe material/ Pipe diameter	C_{hw} 75 mm	C_{hw} 150 mm	C_{hw} 300 mm	C_{hw} 600 mm	C_{hw} 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
Galvanised iron	129	133	—	—	—
Uncoated asbestos cement	142	145	147	150	—
Coated asbestos cement	147	149	150	152	—
Concrete, minimum/maximum values	69/129	79/133	84/138	90/140	95/141
Pre-stressed concrete	—	—	147	150	150
PVC, Brass, Copper, Lead	147	149	150	152	153
Wavy PVC	142	145	147	150	150
Bitumen/cement lined	147	149	150	152	153

Table 3.3. Correction of the Hazen–Williams factors (Bhave, 1991).

C_{hw}	$v < 0.9$ m/s per halving	$v > 0.9$ m/s per doubling
less than 100	+5%	−5%
100–130	+3%	−3%
130–140	+1%	−1%
greater than 140	−1%	+1%

The Manning Equation is more suitable for rough pipes where N is greater than 0.015 $m^{-1/3}s$. It is frequently used for open channel flows rather than pressurised flows.

Comparison of the friction loss equations
The straightforward calculation of pipe resistance, being the main advantage of the Hazen–Williams and Manning equations, has lost its relevance as a result of developments in computer technology. The research also shows some limitations in the application of these equations compared to the Darcy–Weisbach Equation (Liou, 1998). Nevertheless, this is not necessarily a problem for engineering practice and the Hazen–Williams Equation in particular is still widely used in some parts of the world.

Figures 3.9 and 3.10 show the friction loss diagrams for a range of diameters and two roughness values calculated by each of the three equations. The flow in two pipes of different length, L = 200 and 2000 m

Table 3.4. The Manning factors (Bhave, 1991).

Pipe material	N ($m^{-1/3}s$)
PVC, Brass, Lead, Copper, Glass fibre	0.008–0.011
Pre-stressed concrete	0.009–0.012
Concrete	0.010–0.017
Welded steel	0.012–0.013
Coated cast iron	0.012–0.014
Uncoated cast iron	0.013–0.015
Galvanised iron	0.015–0.017

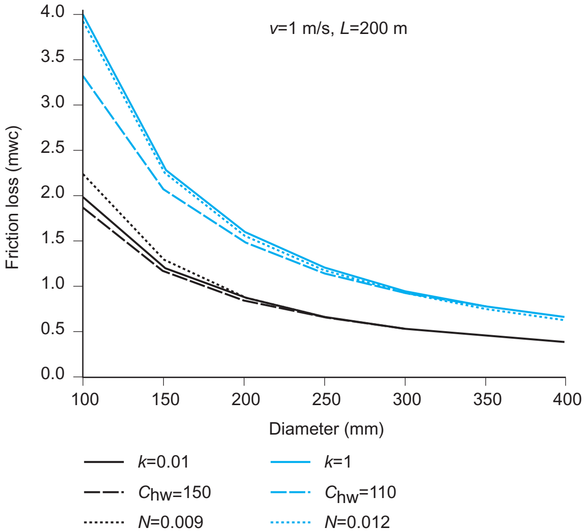


Figure 3.9. Comparison of the friction loss equations: mid range diameters, v = 1 m/s, L = 200 m.

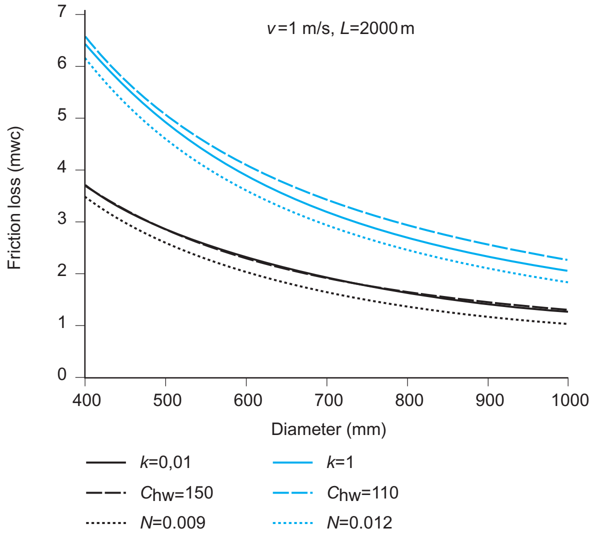


Figure 3.10. Comparison of the friction loss equations: large diameters, $v = 1$ m/s, $L = 2000$ m.

respectively, is determined for velocity $v = 1$ m/s. Thus in all cases, for D in m and Q in m^3/s:

$$Q = v\frac{D^2\pi}{4} = 0.7854D^2$$

The example shows little difference between the results obtained by three different equations. Nevertheless, the same roughness parameters have a different impact on the friction loss in the case of larger and longer pipes.

The difference in results becomes larger if the roughness values are not properly chosen. Figure 3.11 shows the friction loss calculated using the roughness values suggested for PVC in Tables 3.1, 3.2 and 3.4.

Hence, *the choice of a proper roughness value is more relevant than the choice of the friction loss equation itself*. Which of the values fits the best to the particular case can be confirmed only by field measurements. In general, the friction loss will rise when there is:

1 an increase in pipe discharge,
2 an increase in pipe roughness,
3 an increase in pipe length,
4 a reduction in pipe diameter,
5 a decrease in water temperature.

In reality, the situations causing this to happen are:

– higher consumption or leakage,
– corrosion growth,
– network expansion.

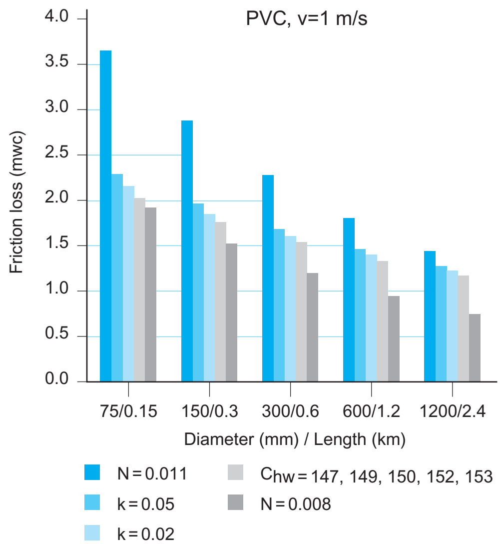


Figure 3.11. Comparison of the friction loss equations for various PVC roughness factors.

Table 3.5. Hydraulic gradient in pipe $D = 300$ mm, $Q = 80$ l/s, $T = 10$ °C.

Parameter	$k = 0.01$ mm	$k = 0.1$ mm	$k = 1$ mm	$k = 5$ mm
S (m/km)	3.3	3.8	6.0	9.9
Increase (%)	–	15	58	65

The friction loss equations clearly point to the pipe diameter as the most sensitive parameter. The Darcy–Weisbach Equation shows that each halving of D (e.g. from 200 to 100 mm) increases the head-loss $2^5 = 32$ times! Moreover, the discharge variation will have a quadratic impact on the head-losses, while these grow linearly with the increase of the pipe length. The friction losses are less sensitive to the change of the roughness factor, particularly in smooth pipes (an example is shown in Table 3.5). Finally, the impact of water temperature variation on the head-losses is marginal.

PROBLEM 3.5

For pipe $L = 450$ m, $D = 300$ mm and flow rate of 120 l/s, calculate the friction loss by comparing the Darcy–Weisbach- ($k = 0.2$ mm), Hazen–Williams- ($C_{hw} = 125$) and Manning equations ($N = 0.01$). The water temperature can be assumed at 10 °C.

If the demand grows at the exponential rate of 1.8% annually, what will be the friction loss in the same pipe after 15 years? The assumed value of an increased absolute roughness in this period equals $k = 0.5$ mm.

Answer:
For a flow Q = 120 l/s and a diameter of 300 mm, the velocity in the pipe:

$$v = \frac{4Q}{D^2\pi} = \frac{4\times0.12}{0.3^2\times3.14} = 1.70\ \text{m/s}$$

Based on the water temperature, the kinematic viscosity can be calculated from Equation 3.22:

$$\upsilon = \frac{497\times10^{-6}}{(T + 42.5)^{1.5}} = \frac{497\times10^{-6}}{(10 + 42.5)^{1.5}} = 1.31\times10^{-6}\ \text{m}^2/\text{s}$$

The Reynolds number then becomes:

$$Re = \frac{vD}{\upsilon} = \frac{1.70\times0.3}{1.31\times10^{-6}} = 3.9\times10^5$$

For the value of relative roughness k/D = 0.2/300 = 0.00067 and the calculated Reynolds number, the friction factor λ can be determined from the Moody diagram in Figure 3.8 ($\lambda \approx 0.019$). Based on the value of the Reynolds number (>> 4000), the flow regime is obviously turbulent. The same result can also be obtained by applying the Barr approximation. From Equation 3.20:

$$\lambda = 0.25 \Big/ \log^2\left[\frac{5.1286}{Re^{0.89}} + \frac{k}{3.7D}\right]$$

$$= 0.25 \Big/ \log^2\left[\frac{5.1286}{(3.9\times10^5)^{0.89}} + \frac{0.2}{3.7\times300}\right] = 0.019$$

Finally, the friction loss from the Darcy–Weisbach Equation is determined as:

$$h_f = \frac{8\lambda L}{\pi^2 g D^5}Q^2 = \frac{\lambda L}{12.1 D^5}Q^2 = \frac{0.019\times450}{12.1\times0.3^5}0.12^2 = 4.18\ \text{mwc}$$

Applying the Hazen–Williams Equation with C_{hw} = 125, the friction loss becomes:

$$h_f = \frac{10.68L}{C_{hw}^{1.852}D^{4.87}}Q^{1.852} = \frac{10.68\times450}{125^{1.852}0.3^{4.87}}0.12^{1.852} = 4.37\ \text{mwc}$$

Introducing a correction for the C_{hw} value of 3%, as suggested in Table 3.3 based on the velocity of 1.7 m/s (almost twice the value of 0.9 m/s), yields a value of C_{hw}, which is reduced to 121. Using the same formula, the friction loss then becomes $h_f = 4.64$ mwc, which is 6% higher than the initial figure.

Finally, applying the Manning Equation with the friction factor $N = 0.01$:

$$h_f = \frac{10.29N^2L}{D^{16/3}}Q^2 = \frac{10.29 \times 0.01^2 \times 450}{0.3^{16/3}}0.12^2 = 4.10 \text{ mwc}$$

With the annual growth rate of 1.8%, the demand after 15 years becomes:

$$Q_{15} = 120\left(1 + \frac{1.8}{100}\right)^{15} = 156.82 \text{ l/s}$$

which, with the increase of the k-value to 0.5 mm, yields the friction loss of 8.60 mwc by applying the Darcy–Weisbach Equation in the same way as shown above. The interim calculations give the following values of the parameters involved: $v = 2.22$ m/s, $Re = 5.1 \times 10^5$ and $\lambda = 0.023$. The final result represents an increase of more than 100% compared to the original value of the friction loss (at the demand increase of approximately 30%).

Self-study:
Workshop problems A1.2.1–A1.2.3 (Appendix 1)
Spreadsheet lessons A5.1.2 and A5.1.3 (Appendix 5)

3.2.2 *Minor losses*

Minor (in various literature *local* or *turbulence*) losses are usually caused by installed valves, bends, elbows, reducers, etc. Although the effect of the disturbance is spread over a short distance, the minor losses are for the sake of simplicity attributed to a cross-section of the pipe. As a result, an instant drop in the hydraulic grade line will be registered at the place of obstruction (see Figure 3.12).
Factors R_m and n_m from Equation 3.15 are uniformly expressed as:

$$R_m = \frac{8\xi}{\pi^2 g D^4} = \frac{\xi}{12.1D^4}; \quad n_m = 2 \tag{3.25}$$

Minor loss coefficient

where ξ represents the *minor (local) loss coefficient*. This factor is usually determined by experiments. The values for most typical appendages are given in Appendix 3. A very detailed overview can be found in Idel'cik (1986).

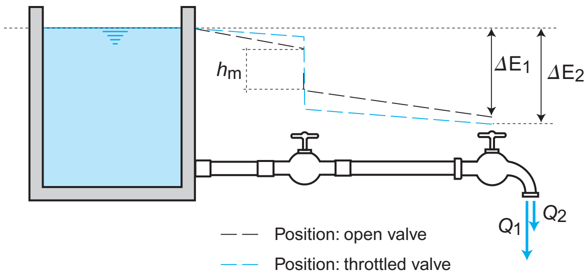


Figure 3.12. Minor loss caused by valve operation.

The minor loss factors for various types of valves are normally supplied together with the device. The corresponding equation may vary slightly from 3.25, mostly in order to enable a diagram that is convenient for easy reading of the values. In the example shown in Figure 3.13, the minor loss of a butterfly valve is calculated in mwc as: $h_m = 10Q^2/K_v^2$, for Q in m^3/h. The K_v-values can be determined from the diagram for different valve diameters and settings.

Substantial minor losses are measured in the following cases:

1 the flow velocity is high, and/or
2 there is a significant valve throttling in the system.

Such conditions commonly occur in pumping stations and in pipes of larger capacities where installed valves are regularly operated; given the magnitude of the head-loss, the term 'minor' loss may not be appropriate in those situations. Within the distribution network on a large scale, the minor losses are comparatively smaller than the friction losses. Their impact on overall head-loss is typically represented through adjustment of the roughness values (increased k and N or reduced C_{hw}). In such cases, $\Delta H \approx h_f$ is an acceptable approximation and the hydraulic gradient then becomes:

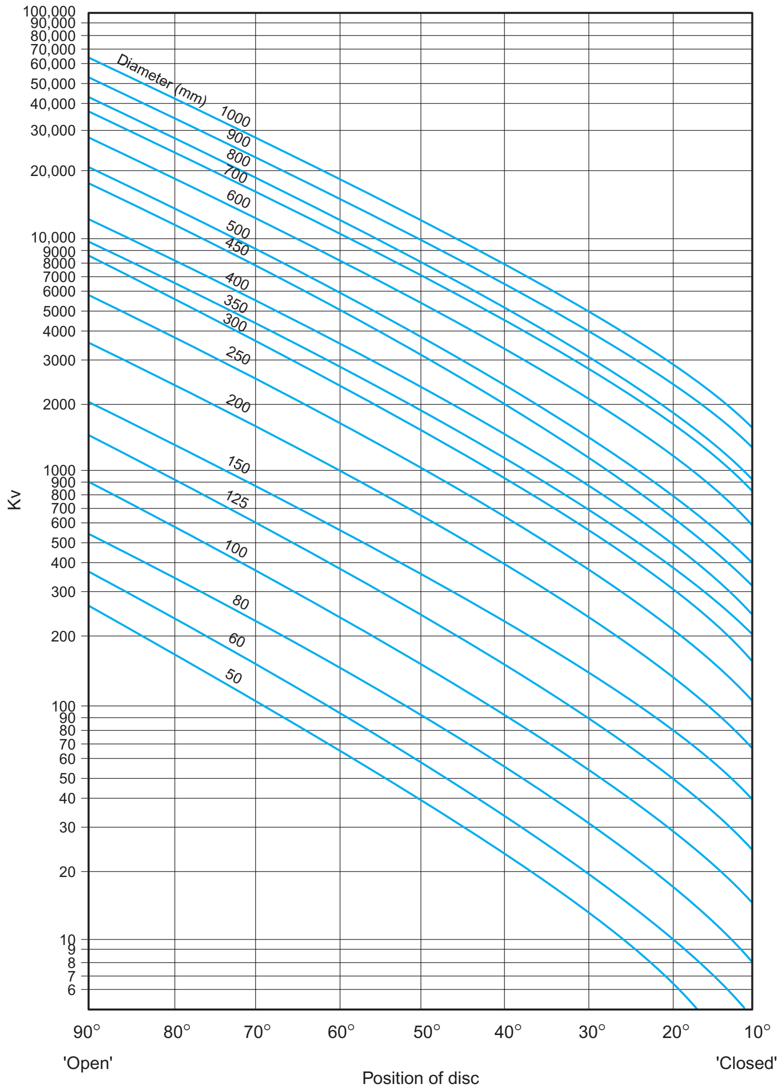
$$S = \frac{\Delta H}{L} \approx \frac{h_f}{L} \qquad (3.26)$$

Equivalent pipe lengths

The other possibility of considering the minor losses is to introduce so-called *equivalent pipe lengths*. This approach is sometimes used for the design of indoor installations where the minor loss impact is simulated by assuming an increased pipe length (for example, up to 30–40%) from the most critical end point.

3.3 SINGLE PIPE CALCULATION

Summarised from the previous paragraph, the basic parameters involved in the head-loss calculation of a single pipe using the



3.13. Example of minor loss diagram from valve operation.

Darcy–Weisbach Equation are:

1 length L,
2 diameter D,
3 absolute roughness k,
4 discharge Q,
5 piezometric head difference ΔH (i.e. the head-loss),
6 water temperature T.

The parameters derived from the above are:

7 velocity, $v = f(Q, D)$,
8 hydraulic gradient, $S = f(\Delta H, L)$,
9 kinematic viscosity, $\upsilon = f(T)$,
10 Reynolds number, $Re = f(v, D, \upsilon)$,
11 friction factor, $\lambda = f(k, D, \mathrm{Re})$.

In practice, three of the six basic parameters are always included as an input:

– L, influenced by the consumers' location,
– k, influenced by the pipe material and its overall condition,
– T, influenced by the ambient temperature.

The other three, D, Q and ΔH, are parameters of major impact on pressures and flows in the system. Any of these parameters can be considered as the overall output of the calculation after setting the other two in addition to the three initial input parameters. The result obtained in such a way answers one of the three typical questions that appear in practice:

1 What is the available head-loss ΔH (and consequently the pressure) in a pipe of diameter D, when it conveys flow Q?
2 What is the flow Q that a pipe of diameter D can deliver if certain maximum head-loss ΔH_{max} (i.e. the minimum pressure p_{min}) is to be maintained?
3 What is the optimal diameter D of a pipe that has to deliver the required flow Q at a certain maximum head-loss ΔH_{max} (i.e. minimum pressure p_{min})?

The calculation procedure in each of these cases is explained below. The form of the Darcy–Weisbach Equation linked to kinetic energy is more suitable in this case:

$$\Delta H \approx h_f = \frac{\lambda L}{12.1 D^5} Q^2 = \lambda \frac{L v^2}{D 2g}; \quad S = \frac{\lambda v^2}{D 2g} \tag{3.27}$$

3.3.1 *Pipe pressure*

The input data in this type of the problem are: L, D, k, Q or v, and T, which yield ΔH (or S) as the result. The following procedure is to be applied:

1 For given Q and D, find out the velocity, $v = 4Q/(D^2\pi)$.
2 Calculate Re from Equation 3.21.
3 Based on the Re value, choose the appropriate friction loss equation, 3.20 or 3.23, and determine the λ-factor. Alternatively, use the Moody diagram for an appropriate k/D ratio.
4 Determine ΔH (or S) by Equation 3.27.

The sample calculation has already been demonstrated in Problem 3.5. To be able to define the pressure head, $p/\rho g$, an additional input is necessary:
– the pipe elevation heads, Z, and
– known (fixed) piezometric head, H, at one side.

There are two possible final outputs for the calculation:
1 If the downstream (discharge) piezometric head is specified, suggesting the minimum pressure to be maintained, the final result will show the required head/pressure at the upstream side i.e. at the supply point.
2 If the upstream (supply) piezometric head is specified, the final result will show the available head/pressure at the downstream side i.e. at the discharge point.

PROBLEM 3.6
The distribution area is supplied through a transportation pipe $L = 750$ m, $D = 400$ mm and $k = 0.3$ mm, with the average flow rate of 1260 m^3/h. For this flow, the water pressure at the end of the pipe has to be maintained at a minimum 30 mwc. What will be the required piezometric level and also the pressure on the upstream side in this situation? The average pipe elevation varies from $Z_2 = 51$ msl at the downstream side to $Z_1 = 75$ msl at the upstream side. It can be assumed that the water temperature is 10 °C.

Answer:
For flow $Q = 1260$ m^3/h $= 350$ l/s and the diameter of 400 mm:

$$v = \frac{4Q}{D^2\pi} = \frac{4\times 0.35}{0.4^2\times 3.14} = 2.79 \text{ m/s}$$

For temperature $T = 10$ °C, the kinematic viscosity from Equation 3.22, $\upsilon = 1.31 \times 10^{-6}$ m^2/s. The Reynolds number takes the value of:

$$Re = \frac{vD}{\upsilon} = \frac{2.79\times 0.4}{1.31\times 10^{-6}} = 8.5\times 10^5$$

and the friction factor λ from Barr's Equation equals:

$$\lambda = 0.25\Big/\log^2\left[\frac{5.1286}{Re^{0.89}} + \frac{k}{3.7D}\right]$$

$$= 0.25\Big/\log^2\left[\frac{5.1286}{(8.5\times 10^5)^{0.89}} + \frac{0.3}{3.7\times 400}\right] \approx 0.019$$

The friction loss from the Darcy–Weisbach Equation can be determined as:

$$h_f = \frac{\lambda L}{12.1 D^5} Q^2 = \frac{0.019 \times 750}{12.1 \times 0.4^5} 0.35^2 \approx 14\,\text{mwc}$$

The downstream pipe elevation is given at $Z_2 = 51$ msl. By adding the minimum required pressure of 30 mwc to it, the downstream piezometric head becomes $H_2 = 51 + 30 = 81$ msl. On the upstream side, the piezometric head must be higher for the value of calculated friction loss, which produces a head of $H_1 = 81 + 14 = 95$ msl. Finally, the pressure on the upstream side will be obtained by deducting the upstream pipe elevation from this head. Hence $p_1/\rho g = 95 - 75 = 20$ mwc. Due to configuration of the terrain in this example, the upstream pressure is lower than the downstream one. For the calculated friction loss, the hydraulic gradient $S = h_f/L = 14/750 \approx 0.019$.

3.3.2 *Maximum pipe capacity*

For determination of the maximum pipe capacity, the input data are: L, D, k, ΔH (or S), and T. The result is flow Q.
Due to the fact that the λ-factor depends on the Reynolds number i.e. the flow velocity that is not known in advance, an iterative procedure is required here. The following steps have to be executed:

1. Assume the initial velocity (usually, $v = 1$ m/s).
2. Calculate Re from Equation 3.21.
3. Based on the Re value, choose the appropriate friction loss equation, 3.20 or 3.23, and calculate the λ-factor. For selected Re- and k/D values, the Moody diagram can also be used as an alternative.
4. Calculate the velocity after re-writing Equation 3.27:

$$v = \sqrt{\frac{2gDS}{\lambda}} \tag{3.28}$$

If the values of the assumed and determined velocity differ substantially, steps 2–4 should be repeated by taking the calculated velocity as the new input. When a sufficient accuracy has been reached, usually after 2–3 iterations for flows in the transitional turbulence zone, the procedure is completed and the flow can be calculated from the final velocity. If the flow is in the rough turbulence zone, the velocity obtained in the first iteration will already be the final one, as the calculated friction factor will remain constant (being independent from the value of the Reynolds number).

If the Moody diagram is used, an alternative approach can be applied for determination of the friction factor. The calculation starts by assuming the rough turbulence regime:

1. Read the initial λ value from Figure 3.8 based on the k/D ratio (or calculate it by applying Equation 3.24).

2 Calculate the velocity by applying Equation 3.28.
3 Calculate *Re* from Equation 3.21.

Check on the graph if the obtained Reynolds number corresponds to the assumed λ and k/D. If not, read the new λ-value for the calculated Reynolds number and repeat steps 2 and 3. Once a sufficient accuracy for the λ-value has been reached, the velocity calculated from this value will be the final velocity.

Both approaches are valid for a wide range of input parameters. The first one is numerical, i.e. suitable for computer programming. The second one is simpler for manual calculations; it is shorter and avoids estimation of the velocity in the first iteration. However, this approach relies very much on accurate reading of the values from the Moody diagram.

PROBLEM 3.7
For the system from Problem 3.6 (Section 3.1.1), calculate the maximum capacity that can be conveyed if the pipe diameter is increased to $D = 500$ mm and the head-loss has been limited to 10 m per km of the pipe length. The roughness factor for the new pipe diameter can be assumed at $k = 0.1$ mm.

Answer:
Assume velocity $v = 1$ m/s. For the temperature $T = 10°C$, the kinematic viscosity from Equation 3.22, $\upsilon = 1.31 \times 10^{-6}$ m^2/s. With diameter $D = 500$ mm, the Reynolds number takes the value of:

$$Re = \frac{vD}{\upsilon} = \frac{1\times 0.5}{1.31\times 10^{-6}} = 3.8\times 10^5$$

and the friction factor λ from Barr's Equation equals:

$$\lambda = 0.25 \Big/ \log^2\left[\frac{5.1286}{\mathrm{Re}^{0.89}} + \frac{k}{3.7D}\right]$$
$$= 0.25 \Big/ \log^2\left[\frac{5.1286}{(3.8\times 10^5)^{0.89}} + \frac{0.1}{3.7\times 500}\right] \approx 0.016$$

The new value of the velocity based on the maximum-allowed hydraulic gradient $S_{max} = 10/1000 = 0.01$ is calculated from Equation 3.28:

$$v = \sqrt{\frac{2gDS}{\lambda}} = \sqrt{\frac{2\times 9.81\times 0.5\times 0.01}{0.016}} = 2.48 \text{ m/s}$$

The result differs substantially from the assumed velocity and the calculation should be repeated in the second iteration with this value as a new assumption. Hence:

$$Re = \frac{2.48 \times 0.5}{1.31 \times 10^{-6}} = 9.5 \times 10^5$$

and the friction factor λ equals:

$$\lambda = 0.25 \Big/ \log^2\left[\frac{5.1286}{(9.5 \times 10^5)^{0.89}} + \frac{0.1}{3.7 \times 500}\right] \approx 0.015$$

The new resulting velocity will be:

$$v = \sqrt{\frac{2 \times 9.81 \times 0.5 \times 0.01}{0.015}} = 2.57 \text{ m/s}$$

which can be considered as a sufficiently accurate result, as any additional iteration that can be done is not going to change this value. Finally, the maximum flow that can be discharged at $S = 0.01$equals:

$$Q = v\frac{D^2\pi}{4} = 2.57\frac{0.5^2 \times 3.14}{4} \approx 0.5\text{m}^3/\text{s} \approx 1800\text{m}^3/\text{h}$$

In the alternative approach, the initial λ value assuming the rough turbulent zone can be read from the Moody diagram in Figure 3.8. For a value of $k/D = 0.1/500 = 0.0002$, it is approximately 0.014. The calculation from the rewritten Equation 3.24 gives:

$$\lambda = 0.25 \Big/ \log^2\left[\frac{k}{3.7D}\right] = 0.25 \Big/ \log^2\left[\frac{0.1}{3.7 \times 500}\right] = 0.0137$$

With this value:

$$v = \sqrt{\frac{2gDS}{\lambda}} = \sqrt{\frac{2 \times 9.81 \times 0.5 \times 0.01}{0.137}} = 2.66 \text{ m/s}$$

and the Reynolds number then becomes:

$$Re = \frac{2.66 \times 0.5}{1.31 \times 10^{-6}} = 1.0 \times 10^6$$

which means that the new reading for λ is closer to the value of 0.015 ($k/D = 0.0002$). Repeated calculation of the velocity and the Reynolds number with this figure leads to a final result as in the first approach.

Self-study:
Workshop problems A1.2.4–A1.2.7 (Appendix 1)
Spreadsheet lesson A5.1.4 (Appendix 5)

3.3.3 *Optimal diameter*

In the calculation of optimal diameters, the input data are: L, k, Q, ΔH (or S), and T. The result is diameter D.

The iteration procedure is similar to the one in the previous case, with the additional step of calculating the input diameter based on the assumed velocity:

1 Assume the initial velocity (usually, $v = 1$ m/s).
2 Calculate the diameter from the velocity/flow relation. $D^2 = 4Q/(v\pi)$.
3 Calculate Re from Equation 3.21.
4 Based on the Re value, choose the appropriate friction loss equation, 3.20 or 3.23, and determine the λ-factor. For selected Re- and k/D values, the Moody diagram can also be used instead.
5 Calculate the velocity from Equation 3.28.

If the values of the assumed and determined velocity differ substantially, steps 2–5 should be repeated by taking the calculated velocity as the new input.

After a sufficient accuracy has been achieved, the calculated diameter can be rounded up to a first higher (manufactured) size.

This procedure normally requires more iterations than for the calculation of the maximum pipe capacity. The calculation of the diameter from an assumed velocity is needed as the proper diameter assumption is often difficult and an inaccurate guess of D accumulates more errors than in the case of the assumption of velocity. For those reasons, the second approach in Section 3.3.2 is not recommended in this case.

PROBLEM 3.8
In case the flow from the previous problem has to be doubled to $Q = 3600$ m^3/h, calculate the diameter that would be sufficient to convey it without increasing the hydraulic gradient. The other input parameters remain the same as in Problem 3.7 (Section 3.3.2).

Answer:
Assume velocity $v = 1$ m/s. Based on this velocity, the diameter D:

$$D = 2\sqrt{\frac{Q}{v\pi}} = 2\times\sqrt{\frac{1}{1\times 3.14}} = 1.128 \text{ m}$$

and the Reynolds number:

$$Re = \frac{vD}{\upsilon} = \frac{1\times 1.128}{1.31\times 10^{-6}} = 8.6\times 10^5$$

$$Re = \frac{vD}{\upsilon} = \frac{1 \times 1.128}{1.31 \times 10^{-6}} = 8.6 \times 10^5$$

The friction factor λ from Barr's Equation equals:

$$\lambda = 0.25 \Big/ \log^2\left[\frac{5.1286}{Re^{0.89}} + \frac{\text{k}}{3.7\text{D}}\right]$$
$$= 0.25 \Big/ \log^2\left[\frac{5.1286}{(8.6 \times 10^5)^{0.89}} + \frac{0.1}{3.7 \times 1128}\right] \approx 0.0135$$

and at $S_{max} = 0.01$ the velocity from Equation 3.28 becomes:

$$v = \sqrt{\frac{2gDS}{\lambda}} = \sqrt{\frac{2 \times 9.81 \times 1.128 \times 0.01}{0.0135}} = 4.04 \text{ m/s}$$

The result is substantially different than the assumed velocity and the calculation has to be continued with several more iterations. The results after applying the same procedure are shown in the following table:

Iter.	v_{ass} (m/s)	D(mm)	Re(-)	λ(-)	v_{calc} (m/s)
2	4.04	561	1.7×10^6	0.0141	2.79
3	2.79	676	1.4×10^6	0.0139	3.09
4	3.09	642	1.5×10^6	0.0139	3.01
5	3.01	650	1.5×10^6	0.0139	3.03

with the final value for the diameter of $D = 650$ mm. The manufactured size would be, say, $D = 700$ mm.

Self-study:
Workshop problem A1.2.8 (Appendix 1)
Spreadsheet lesson A5.1.5 (Appendix 5)

3.3.4 *Pipe charts and tables*

Straightforward determination of the required pressures, flows or diameters is possible by using the *pipe charts* or *pipe tables*. These are created by combining the Darcy–Weisbach and Colebrook–White Equations. Substituting λ and Re in Equation 3.19, by using Equations 3.28 and 3.21 respectively, yields the following equation:

$$v = -2\sqrt{2gDS}\log\left[\frac{2.51\upsilon}{D\sqrt{2gDS}} + \frac{k}{3.7D}\right] \tag{3.29}$$

For a fixed k-value and the water temperature (i.e. the viscosity), the velocity is calculated for common ranges of D and S. The values for v, D, S and Q are then plotted or sorted in a tabular form (see Appendix 4).

The chart in Figure 3.14 shows an example of a flow rate of 20 l/s (top axis) passing through a pipe of diameter $D = 200$ mm (bottom axis). From the intersection of the lines connecting these two values it emerges that the corresponding velocity (left axis) and hydraulic gradient (right axis) would be around 0.6 m/s and 2 m/km, respectively. The same flow rate in a pipe $D = 300$ mm yields much lower values: the velocity would be below 0.3 m/s and the gradient around 0.3 m/km.

It is important to note that the particular graph or table is valid for *one single* roughness value and *one single* water temperature. Although the variation of these parameters has a smaller effect on the friction loss than the variation of D, v or Q, this limits the application of the tables and graphs if the values specifically for k differ substantially from those used in the creation of the table/graph. As an example, Table 3.6 shows the difference in the calculation of hydraulic gradients for the range of values for k and T.

In former times, the pipe charts and tables were widely used for hydraulic calculations. Since the development of PC-spreadsheet programmes, their relevance has somewhat diminished. Nevertheless, they

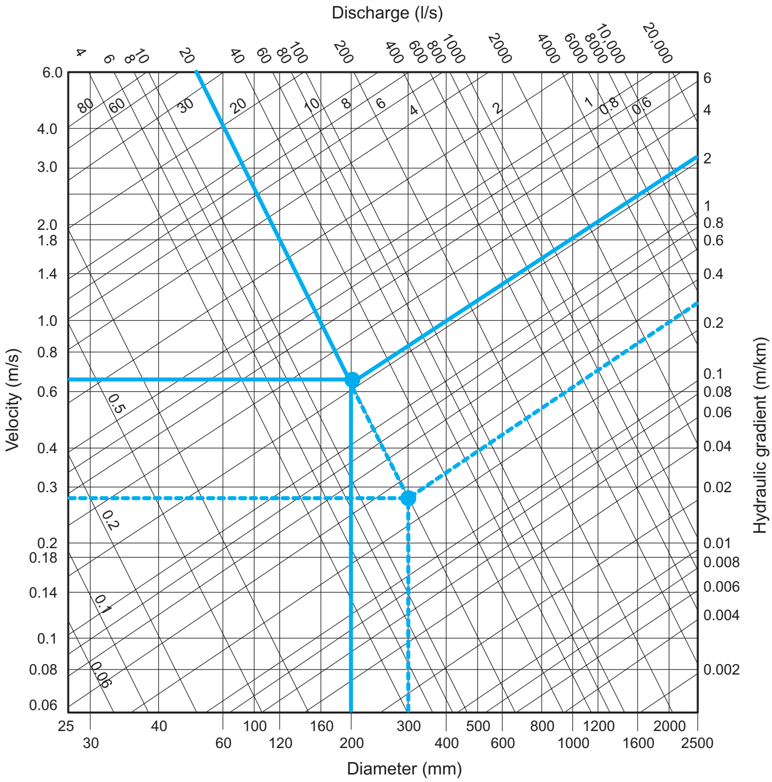


Figure 3.14. Example of a pipe chart.

Table 3.6. Hydraulic gradient S (-) in pipe D = 400 mm at Q = 200 l/s.

Parameter	k = 0.01 mm	k = 0.1 mm	k = 1 mm	k = 5 mm
T = 10 °C	0.0044	0.0052	0.0082	0.0133
T = 20 °C	0.0042	0.0051	0.0081	0.0132
T = 40 °C	0.0040	0.0049	0.0081	0.0132

are a useful help in providing quick and straightforward estimates of pipe discharges for given design layouts.

PROBLEM 3.9
Using the pipe tables, determine the maximum discharge capacity for pipe D = 800 mm for the following roughness values: k = 0.01, 0.5, 1 and 5 mm and the maximum-allowed hydraulic gradients of S = 0.001, 0.005, 0.01 and 0.02, respectively. The water temperature can be assumed at T = 10°C.

Answer:
The following table shows the results read for pipe D = 800 mm from the tables in Appendix 4:

Discharge flows Q (l/s) for pipe D = 800 mm (for T = 10°C)

Parameter	k = 0.01 mm	k = 0.5 mm	k = 1 mm	k = 5 mm
S = 0.001	559.5	465.6	432.3	348.3
S = 0.005	1336.2	1052.5	972.8	780.0
S = 0.01	1936.8	1492.6	1377.8	1103.6
S = 0.02	2800.1	2115.1	1950.6	1561.1

The results suggest the following two conclusions:
1 For fixed values of S, the discharge capacity is reduced by the increase of the roughness value. In other words, the pipes start to loose their conveying capacity as they get older, which is reflected in reality by the drop of demand and/or pressure.
2 The discharge at the fixed k-value will increase by allowing the higher hydraulic gradient. In other words, if more of a friction loss is allowed in the network, more water will be distributed but at higher operational costs (because of additional pumping).

3.3.5 *Equivalent diameters*

During planning of network extensions or renovations, the alternative of laying single pipe or pipes connected in parallel or series is sometimes compared. To provide a hydraulically equivalent system, *the capacity and hydraulic gradient along the considered section should remain* *unchanged* in all options. Those pipes are then of *equivalent diameters* (see Figure 3.15).

Each pipe in the parallel arrangement creates the same friction loss, which is equal to the total loss at the section. The total capacity is the sum of the flows in all pipes. Hence, for n pipes it is possible to write:

$$\Delta H_{\mathrm{equ}} = \Delta H_1 = \Delta H_2 = \cdots = \Delta H_n$$

$$Q_{\mathrm{equ}} = Q_1 + Q_2 + \cdots + Q_n$$

Pipes in parallel are more frequently of the same diameter, allowing for easier maintenance and handling of irregular situations. Furthermore, they will often be laid in the same trench i.e. along the same route and can therefore be assumed to be of the same length in which case the slope of the hydraulic grade line for all pipes will be equal. Nevertheless, the equation $S_{\mathrm{equ}} = S_1 = S_2 \cdots = S_{\mathrm{n}}$ is not always true as the pipes connected in parallel need not necessarily be of identical length.

For pipes in series, the basic hydraulic condition is that each pipe carries the same flow rate. The total energy loss is the sum of the losses in all pipes. If written for n pipes:

$$\Delta H_{\mathrm{equ}} = \Delta H_1 + \Delta H_2 + \cdots + \Delta H_n$$

$$Q_{\mathrm{equ}} = Q_1 = Q_2 = \cdots = Q_n$$

Equation $S_{\mathrm{equ}} = S_1 + S_2 + \cdots + S_n$, will not normally be true except in the hypothetical case of $S_1 = S_2 = \cdots = S_n$.

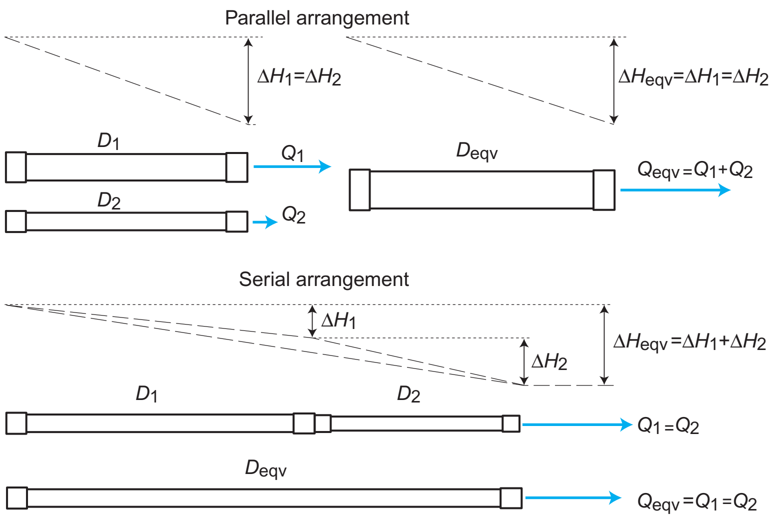


Figure 3.15. Equivalent diameters.

The hydraulic calculation of the equivalent diameters further proceeds based on the principles of the single pipe calculation, as explained in Paragraph 3.3.

PROBLEM 3.10
A pipe $L = 550$ m, $D = 400$ mm, and $k = 1$ mm transports the flow of 170 l/s. By an extension of the system this capacity is expected to grow to 250 l/s. Two alternatives to solve this problem are considered:
1 To lay a parallel pipe of smaller diameter on the same route, or
2 To lay a parallel pipe of the same diameter on a separate route with a total length $L = 800$ m.

Using the hydraulic tables for water temperature $T = 10°$C:
a Determine the diameter of the pipe required to supply the surplus capacity of 80 l/s in the first alternative,
b Determine the discharge of the second pipe $D = 400$ mm in the second alternative.

In both cases, the absolute roughness of the new pipes can be assumed to be $k = 0.1$ mm.

Answers:
In the hydraulic tables in Appendix 4 (for $T = 10°$C), the diameter $D = 400$ mm conveys the flow $Q = 156.6$ l/s for the hydraulic gradient $S = 0.005$ and $Q = 171.7$ l/s for $S = 0.006$. Assuming linear interpolation (which introduces negligible error), the flow of 170 l/s will be conveyed at $S = 0.0059$, leading to a friction loss $h_f = S \times L = 0.0059 \times 550 = 3.25$ mwc. This value is to be maintained in the design of the new parallel pipe.

Laying the second pipe in the same trench (i.e. with the same length) should provide an additional flow of 80 l/s. From the hydraulic tables for $k = 0.1$ mm the following closest discharge values can be read:

Discharge flows (l/s) for pipe $k = 0.1$ mm

Parameter	$D = 250$ mm	$D = 300$ mm
$S = 0.005$	57.2	92.5
$S = 0.006$	62.9	101.7

which suggests that the manufactured diameter of 300 mm is the final solution. The flow rate to be conveyed at $S = 0.0059$ would be $Q = 100.8$ l/s (after interpolation) leading to a total supply capacity of 270.8 l/s.

In the second case, the parallel pipe $D = 400$ mm follows an alternative route with a total length of $L = 800$ m. The value of the hydraulic gradient will be consequently reduced to $S = 3.25 / 800 = 0.0041$. The hydraulic tables give the following readings closest to this value:

Discharge flows (l/s) for pipe $k = 0.1$ mm

Parameter	$D = 400$ mm
$S = 0.004$	175.6
$S = 0.005$	197.3

Despite the longer route, this pipe is sufficiently large to convey capacities far beyond the required 80 l/s. For $S = 0.0041$, discharge $Q = 177.8$ l/s and the total supplying capacity from both pipes equals 347.8 l/s. Hence, more water but at higher investment costs.

Self-study:
Workshop problems A1.2.9–A1.2.11 (Appendix 1)
Spreadsheet lessons A5.2.1a–A5.2.5 (Appendix 5)

3.4 SERIAL AND BRANCHED NETWORKS

Calculation of serial and branched networks is entirely based on the methods used for single pipes. The differences in hydraulic performance occur between the branched systems with one supply point and those that have more than one supply point.

3.4.1 *Supply at one point*

With known nodal demands, the flows in all pipes can easily be determined by applying the Continuity Equation (Equation 3.5), starting from the end points of the system (Figure 3.16).

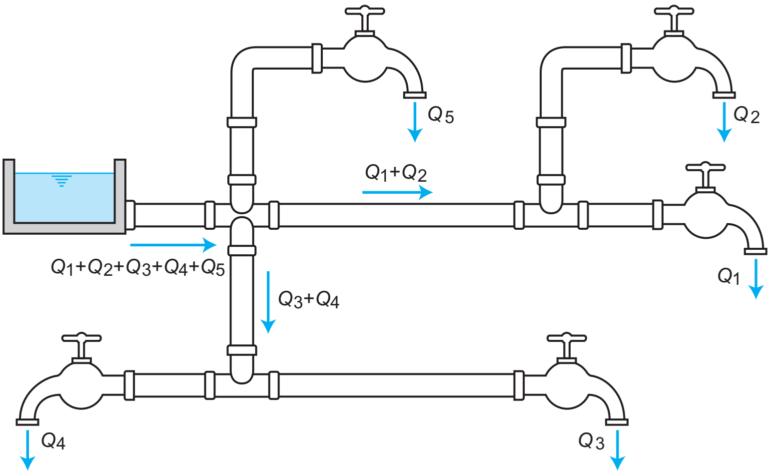


Figure 3.16. Branched network with a single supply point.

If the diameters of the pipes are also known, the head-loss calculation follows the procedure in Section 3.3.1, resulting in the hydraulic gradient *S* for each pipe. In the next step the piezometric heads, and consequently the pressures, will be calculated for each node starting from the node assumed to have the minimum pressure. In this respect potentially critical nodes are those with either high elevation and/or nodes located faraway from the source. Adding or subtracting the head-losses for each pipe, depending on the flow direction, will determine all other heads including the required piezometric head at the supply point. Calculation of the piezometric heads in the opposite direction, starting from the known value at the source, is also possible; this shows the pressures in the system available for specified head at the supplying point.

In situations where pipe diameters have to be designed, the maximum-allowed hydraulic gradient must be included in the calculation input. The iterative procedure from Section 3.3.3 or the pipe charts/tables are required here, leading to actual values of the hydraulic gradient for each pipe based on the best available (manufactured) diameter. Finally, the pressures in the system will also be determined either by setting the minimum pressure criterion or the head available at the supply point.

3.4.2 *Supply at several points*

For more than one supply point, the contribution from each source may differ depending on its piezometric head and distribution of nodal demands in the system. In this case, flows in the pipes connecting the sources are not directly known from the Continuity Equation. These flows can change their rate and even reverse the direction based on the variation of nodal demands. Figures 3.17 and 3.18 show an example of anticipated demand increase in node one.

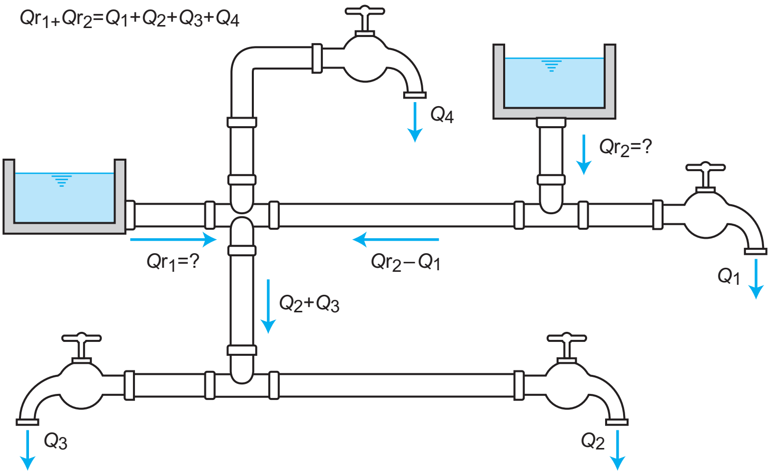


Figure 3.17. Branched network with two supply points.

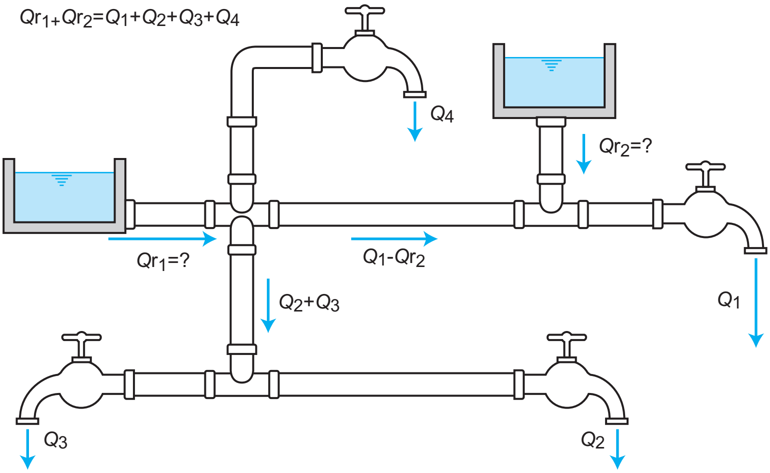
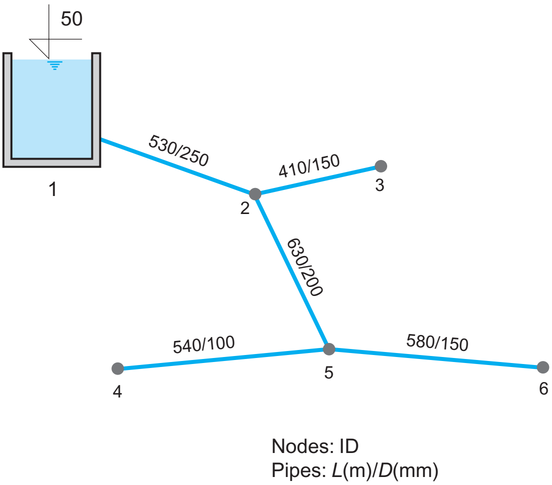


Figure 3.18. Branched network with two supply points, showing an increase of nodal flow Q_1.

Except for the chosen source, fixed conditions are required for all other sources existing in the system: a head, discharge or the hydraulic gradient of the connecting pipe(s). For the remainder, the calculation proceeds in precisely the same manner as in the case of one supply point.

PROBLEM 3.11

For the following branched system, calculate the pipe flows and nodal pressures for surface level in the reservoir of $H = 50$ msl. Assume for all pipes $k = 0.1$ mm, and water temperature of 10°C.



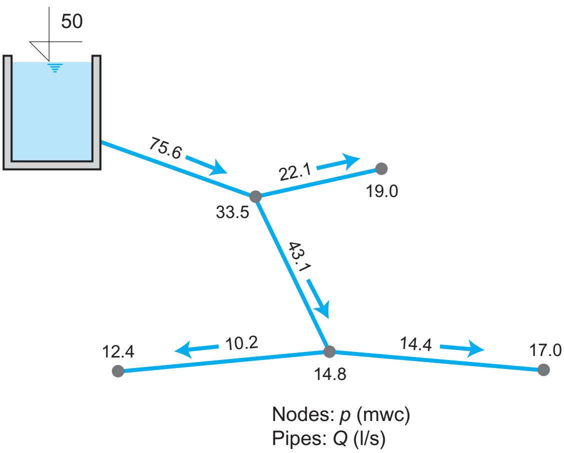
	1	2	3	4	5	6
Z (msl)	—	12	22	17	25	20
Q (l/s)	−75.6	10.4	22.1	10.2	18.5	14.4

Answer:
The total supply from the reservoir equals the sum of all nodal demands, which is 75.6 l/s. Applying the Continuity Equation in each node (Equation 3.5), both the flow rate and its direction can be determined; each pipe conveys the flow that is the sum of all downstream nodal demands. The pipe friction loss will be further calculated by the approach discussed in Problem 3.6 (Section 3.3.1). If the hydraulic tables from Appendix 4 are used, the friction loss will be calculated from interpolated hydraulic gradients at a given diameter and flow rate (for fixed *k* and *T*). The results of the calculation applying the Darcy–Weisbach Equation are shown in the following table.

Pipe	*D* (mm)	*Q* (l/s)	*v* (m/s)	*Re* (-)	λ (-)	*S* (-)	*L* (m)	h_f (mwc)
1–2	250	75.6	1.54	2.9×10^5	0.018	0.0086	530	4.55
2–3	150	22.1	1.25	1.4×10^5	0.020	0.0108	410	4.43
2–5	200	43.1	1.37	2.1×10^5	0.19	0.0090	630	5.70
5–4	100	10.2	1.30	9.9×10^4	0.022	0.0192	540	10.38
5–6	150	14.4	0.81	9.4×10^4	0.021	0.0048	580	2.78

Finally, the pressure in each node is calculated by subtracting the friction losses starting from the reservoir surface level and further deducting the nodal elevation from the piezometric heads obtained in this way. The final results are shown in the following table and figure.

	1	2	3	4	5	6
H(msl)	50	45.45	41.02	29.37	39.75	36.96
p (mwc)	—	33.45	19.02	12.37	14.75	16.96



The lowest pressure appears to be in node 4 (12.4 mwc) resulting from a relatively small diameter (causing large friction loss) of pipe 5–4.

Self-study:
Workshop problems A1.3.1–A1.3.5 (Appendix 1)
Spreadsheet lessons A5.3.1–A5.3.2 (Appendix 5)

3.5 LOOPED NETWORKS

Kirchoff's Laws

The principles of calculation as applied to single pipes are not sufficient in case of looped networks. Instead, a system of equations is required which can be solved by numerical algorithm. This system of equations is based on the analogy with two electricity laws known in physics as *Kirchoff's Laws*. Translated to water distribution networks, these laws state that:

1 The sum of all ingoing and outgoing flows in each node equals zero ($\Sigma Q_i = 0$).
2 The sum of all head-losses along pipes that compose a complete loop equals zero ($\Sigma \Delta H_i = 0$).

The first law is essentially the mass conservation law, resulting in the Continuity Equation that must be valid for each node in the system.

From the second law, it emerges that the hydraulic grade line along one loop is also continuous, just as the flow in any node is. The number of equations that can be formulated applying this law equals the number of loops. For example, in the simple network from Figure 3.19 in the clockwise direction, this yields:

$$(H_r - H_1) + (H_1 - H_2) + (H_2 - H_3) - (H_r - H_3) = 0$$

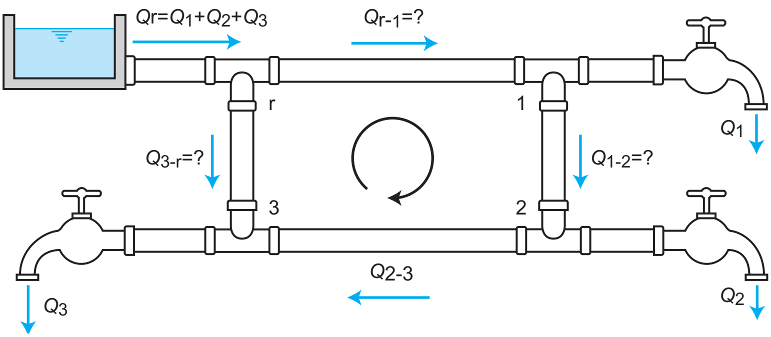


Figure 3.19. Looped network.

3.5.1 *Hardy Cross methods*

Two similar iterative procedures can be derived from Kirchoff's Laws:
1 The method of balancing heads.
2 The method of balancing flows.

These methods, known in literature under the name of Hardy Cross (published in 1936 and developed further by Cornish in 1939), calculate the pipe flows and nodal piezometric heads in looped systems for a given input, which is:
– for pipes: length L, diameter D, absolute roughness k and minor loss factor ξ,
– for nodes: nodal discharge Q and elevation Z.

Pressure in at least one node has to be fixed, which will influence the pressure in the rest of the system. This is usually a supply point.

Successive calculation of the loops (nodes) is executed by following the following steps:

Method of balancing heads
1 Flows from an initial guess are assigned to each pipe. However, these must satisfy the Continuity Equation in all nodes.
2 Head-loss in each pipe is calculated starting from Equation 3.15.
3 The sum of the head-losses along each loop is checked.
4 If the head-loss sum at any loop is outside of the required accuracy range, $0 \pm \varepsilon_{\Delta H}$ mwc, the following flow correction has to be introduced for each pipe within that loop (total n pipes):

$$\delta Q_j = \frac{-\sum_{j=1}^{n} \Delta H_j}{2\sum_{j=1}^{n} |\Delta H_j / Q_j|} \tag{3.30}$$

5 Correction δQ is applied throughout the loop taking consistent orientation: clockwise or anti-clockwise. This has implications for the value of the pipe flows, which will be negative if their direction counters the adopted orientation. The positive/negative sign of the correction should also be taken into account while adding it to the current pipe flow.
6 The iteration procedure is carried out for the new flows, $Q + \delta Q$, repeated in steps 2–5, until $\varepsilon_{\Delta H}$ is satisfied for all loops.
7 After the iteration of flows and head-losses is completed, the pressures in the nodes can be determined from the reference node with fixed pressure, taking into account the flow directions.

The calculation proceeds simultaneously for all loops in the network, with their corresponding corrections δQ being applied in the same iteration. In case of the pipes shared between the two neighbouring loops, the sum of the two δQ-corrections should be applied. The flow continuity in the nodes will not be affected in this case; assuming uniform orientation for both loops will reverse the sign of the composite δQ in one of them.

If the system is supplied from more than one source, the number of unknowns increases. Dummy loops have to be created by connecting the sources with dummy pipes of fictitious L, D and k, but with a fixed ΔH equal to the surface elevation difference between the connected reservoirs. This value has to be maintained throughout the entire iteration process.

Method of balancing flows

1 The estimated piezometric heads are initially assigned to each node in the system, except for the reference i.e. fixed pressure node(s). An arbitrary distribution is allowed in this case.
2 The piezometric head difference is determined for each pipe.
3 Flow in each pipe is determined starting from the head-loss Equation 3.15.
4 The Continuity Equation is checked in each node.
5 If the sum of flows in any node is out of the requested accuracy range, $0 \pm \varepsilon_Q$ m^3/s, the following piezometric head correction has to be introduced in that node (n is the number of pipes connected in the node):

$$\delta H_i = \frac{2\sum_{i=1}^{n} \Delta Q_i}{\sum_{i=1}^{n} |\Delta Q_i / \Delta H_i|} \tag{3.31}$$

6 The iteration procedure is continued with the new heads, $H + \delta H$, repeated in steps 2–5, until ε_Q is satisfied for all nodes.

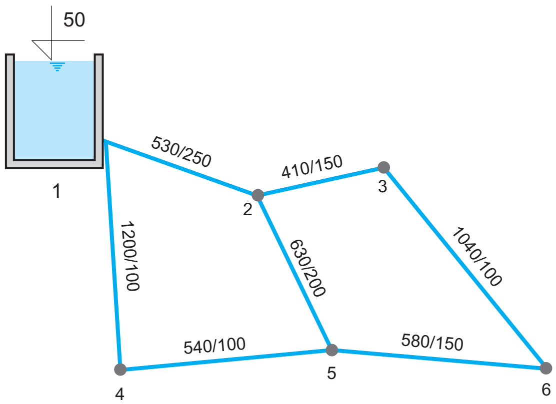
Unlike in the method of balancing heads, faster convergence in the method of balancing flows is reached by applying the corrections consecutively. As a consequence, the flow continuity in some nodes will include the pipe flows calculated from the piezometric heads of the surrounding nodes from the same iteration.

The required calculation time for both methods is influenced by the size of the network. The balancing head method involves systems with a smaller number of equations, equal to the number of loops, which saves time while doing the calculation manually. The balancing flow method requires a larger system of equations, equal to the number of nodes. However, this method excludes the identification of loops, which is of some advantage for computer programming.

The layout and operation of the system may have an impact on convergence in both methods. In general, faster convergence is reached by the balancing head method.

The Hardy Cross methods were widely used in the pre-computer era. The first hydraulic modelling software in water distribution was also based on these methods. Several modifications have been introduced in the meantime. The balancing flow method was first developed for computer applications while the balancing head method still remains a preferred approach for manual calculations of simple looped networks. Both methods are programmable in spreadsheet form, which helps in reducing the calculation time in such cases.

PROBLEM 3.12
To improve the conveyance of the system from Problem 3.11, nodes one and four as well as nodes three and six have been connected with pipes $D = 100$ mm and $L = 1200$ and 1040 m, respectively ($k = 0.1$ mm in both cases). Calculate the pipe flows and nodal pressures for such a system by applying the balancing head method.



Nodes: ID
Pipes: L(m)/D(mm)

Answer:
Two loops are created from the branched system after adding the new pipes. The calculation starts by distributing the pipe flows arbitrarily, but satisfying the Continuity Equation in each node. The next step is to calculate the friction losses in each loop, as the following tables show (negative values mean the reverse direction, from the right node to the left one):

Loop one – Iteration one

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
1–2	250	50.6	1.03	2.12
2–5	200	20.2	0.64	1.36
5–4	100	−14.8	−1.88	−21.17
4–1	100	−25.0	−3.18	−129.89

The sum of all friction losses, which should be equal to 0 for correct flow rate values, is in this case $\Sigma h_f = -147.59$ mwc (selecting the clockwise direction). Thus, the correction of all pipe flows in Loop One will be required in the new iteration. From Equation 3.30 (for $\Delta H = h_f$) this correction becomes $\delta Q = 10.96$ l/s.

In the case of Loop Two:

Loop two – Iteration one

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f(mwc)
5–2	200	−20.20	−0.64	−1.36
2–3	150	20.00	1.13	3.66
3–6	100	−2.10	−0.27	−1.06
6–5	150	−16.50	−0.93	−3.60

The sum of all friction losses in this case is $\Sigma h_f = -2.35$ mwc, which is closer to the final result but also requires another flow correction. After applying Equation 3.30, $\delta Q = 1.21$ l/s.

In the second iteration, the following results were achieved after applying the pipe flows $Q + \delta Q$:

Loop one – Iteration two

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
1–2	250	61.56	1.25	3.07
2–5	200	29.95	0.95	2.85
5–4	100	−3.84	−0.49	−1.66
4–1	100	−14.04	−1.79	−42.53

$\Sigma h_f = -38.27$ mwc and therefore $\delta Q = 5.31$ l/s.

Loop two – Iteration two

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
5–2	200	−29.95	−0.95	−2.85
2–3	150	21.21	1.20	4.10
3–6	100	−0.89	−0.11	−0.23
6–5	150	−15.29	−0.87	−3.12

$\Sigma h_f = -2.10$ mwc and therefore $\delta Q = 1.40$ l/s.

The new flow in pipes 2–5 shared between the loops has been obtained by applying the correction δQ of both loops, i.e. $20.20 + 10.96 - 1.21 = 29.95$ l/s. This pipe in Loop Two has a reversed order of nodes and therefore $Q_{5-2} = -20.20 + 1.21 - 10.96 =- 29.95$ l/s. Hence, the corrected flow of the shared pipe maintains the same value in both loops, once with a positive and once with a negative sign.

In the rest of the calculations:

Loop one – Iteration three

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
1–2	250	66.86	1.36	3.60
2–5	200	33.85	1.08	3.60
5–4	100	1.46	0.19	0.29
4–1	100	−8.74	−1.11	−17.17

$\Sigma h_f = -9.69$ mwc and therefore $\delta Q = 2.09$ l/s.

Loop two – Iteration three

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
5–2	200	−33.85	−1.08	−3.60
2–3	150	22.61	1.28	4.63
3–6	100	0.51	0.06	0.09
6–5	150	−13.89	−0.79	−2.60

$\Sigma h_f = -1.48$ mwc and therefore $\delta Q = 1.11$ l/s.

Loop one – Iteration four

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
1–2	250	68.95	1.40	3.82
2–5	200	34.83	1.11	3.80
5–4	100	3.55	0.45	1.43
4–1	100	−6.65	−0.85	−10.25

$\Sigma h_f = -1.20$ mwc and therefore $\delta Q = 0.29$ l/s.

Loop two – Iteration four

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
5–2	200	−34.83	−1.11	−3.80
2–3	150	23.72	1.34	5.07
3–6	100	1.62	0.21	0.66
6–5	150	−12.78	−0.72	−2.22

$\Sigma h_f = -0.29$ mwc and therefore $\delta Q = 0.16$ l/s.

Loop one – Iteration five

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
1–2	250	69.24	1.41	3.85
2–5	200	34.96	1.11	3.82
5–4	100	3.84	0.49	1.65
4–1	100	−6.36	−0.81	−9.44

$\Sigma h_f = -0.12$ mwc and therefore $\delta Q = 0.03$ l/s.

Loop two – Iteration five

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
5–2	200	−34.96	−1.11	−3.82
2–3	150	23.88	1.35	5.14
3–6	100	1.78	0.23	0.78
6–5	150	−12.62	−0.71	−2.17

$\Sigma h_f = -0.08$ mwc and therefore $\delta Q = 0.04$ l/s.

Loop one – Iteration six

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
1–2	250	69.26	1.41	3.85
2–5	200	34.94	1.11	3.82
5–4	100	3.86	0.49	1.68
4–1	100	−6.34	−0.81	−9.36

$\Sigma h_f = -0.02$ mwc and therefore $\delta Q = 0.00$ l/s.

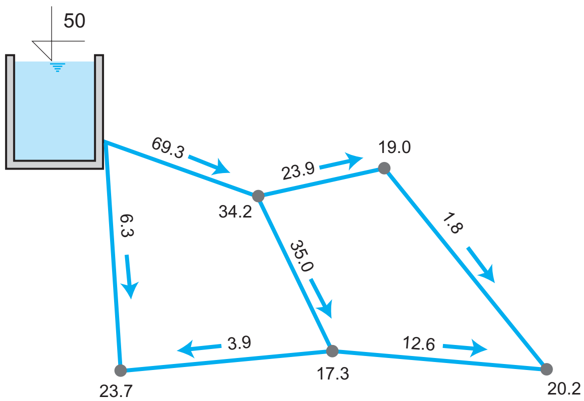
Loop two – Iteration six

Pipe	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)
5–2	200	−34.94	−1.11	−3.82
2–3	150	23.92	1.35	5.15
3–6	100	1.82	0.23	0.82
6–5	150	−12.58	−0.71	−2.16

$\Sigma h_f = -0.01$ mwc and therefore $\delta Q = 0.01$ l/s.

As the tables show, the method already provides fast convergence after the first two to three iterations and continuation of the calculations does not add much to the accuracy of the results while it takes time, specifically in the case of manual calculations.

Determination of the nodal pressures will be done in the same way as in the case of branched systems: starting from the supply point with a fixed piezometric head, or from the point with minimum pressure required in the system. In each pipe, the unknown piezometric head on one side is obtained either by adding the pipe friction loss to the known downstream piezometric head, or by subtracting the pipe friction loss from the known upstream piezometric head. The final results are shown in the following figure:



Nodes: p (mwc)
Pipes: Q (l/s)

Self-study:
Workshop problems A1.4.1–A1.4.3 (Appendix 1)
Spreadsheet lessons A5.4.1 and A5.4.2 (Appendix 5)

3.5.2 *Linear theory*

The Newton Raphson and the Linear Theory methods succeeded the Hardy-Cross methods, as the main approach for solving the non-linear network governing equations. The linear theory method was developed by Wood and Charles, (1972) and involves a remarkably simple linearization compared with the Newton Raphson method. When using the Darcy–Weisbach Equation:

$$\Delta H = (R_f + R_m)|Q|Q = UQ \tag{3.32}$$

where:

$$U = \frac{(\lambda L + \xi D)|Q|}{12.1 D^5} \tag{3.33}$$

The absolute value of Q helps to distinguish between different flow directions (+/− sign).

The following can be written for node i, assuming the inflow to the node has a negative sign (Figure 3.20):

$$Q_i - \sum_{j=1}^{n} Q_{ij} = 0 \tag{3.34}$$

Index n represents the total number of pipes connected to node i, while Q_i is the nodal demand (outflow).

Equation 3.34 is satisfied in the iteration procedure with specified accuracy ε_i, for each node. Thus:

$$Q_i - \sum_{j=1}^{n} Q_{ij} = \pm\varepsilon_i = f(H_i) \tag{3.35}$$

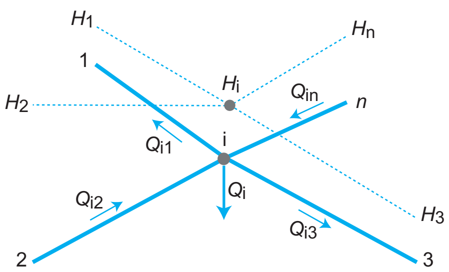


Figure 3.20. Linear theory.

and after combining Equations 3.32 and 3.35:

$$\pm\varepsilon_i = f(H_i) = Q_i - \sum_{j=1}^{n}\frac{H_j}{U_{ij}} + H_i\sum_{j=1}^{n}\frac{1}{U_{ij}} \tag{3.36}$$

The system of linear equations given in 3.36, equals the total number of nodes in the network. The solution of these linear equations can be achieved by any standard procedure (i.e. algorithms for inversion and decomposition). For example, one could apply the Newton Raphson method that would give the nodal head in the $(k + 1)^{\text{th}}$ iteration as follows:

$$H_i^{(k+1)} = H_i^{(k)} - \frac{f(H_i^{(k)})}{f'(H_i^{(k)})} \tag{3.37}$$

Plugging 3.36 into 3.37 yields:

$$f(H_i^{(k)}) = Q_i - \sum_{j=1}^{n}\frac{H_j^{(k)}}{U_{ij}^{(k)}} + H_i^{(k)}\sum_{j=1}^{n}\frac{1}{U_{ij}^{(k)}} \tag{3.38}$$

$$f'(H_i^{(k)}) = \sum_{j=1}^{n}\frac{1}{U_{ij}^{(k)}} \tag{3.39}$$

$$H_i^{(k+1)} = H_i^{(k)} - \left[\frac{Q_i - \sum_{j=1}^{n} H_j^{(k)}/U_{ij}^{(k)} + H_i^{(k)}\sum_{j=1}^{n} 1/U_{ij}^{(k)}}{\sum_{j=1}^{n} 1/U_{ij}^{(k)}}\right] \tag{3.40}$$

Finally, a factor ω that takes values between one and two can be added to improve the convergence:

$$H_i^{(k+1)} = H_i^{(k)}(1 - \omega) + \left[\frac{\sum_{j=1}^{n} H_j^{(k)}/U_{ij}^{(k)} - Q_i}{\sum_{j=1}^{n} 1/U_{ij}^{(k)}}\right]\omega \tag{3.41}$$

Successive Over-Relaxation Method

This method, known as the *Successive Over-Relaxation Method (SOR)*, was recommended by Radojković and Klem (1989). Equation 3.41 shows that the piezometric head in node i depends on:

- piezometric heads at the surrounding nodes $j = 1$ to n, and
- resistance U of the pipes that connect node i with the surrounding nodes.

The size of the equation matrix in this approach is $m \times n_{\max}$, where m is the total number of nodes and $n_{\max}$ is a specified maximum number of nodes connected to each node in the system.

The iteration procedure is executed separately for nodes and pipes and consists of internal and external cycles. The preparation steps are:

1 Setting the initial values of the flow in each pipe. This is usually based on the velocity of 1 m/s and a given pipe diameter.
2 Calculation of the U-values based on the initial flows.
3 Setting the initial values of the piezometric head in each node. As in the case of the Hardy Cross methods, at least one node must be chosen as a reference (fixed head) node, to allow determination of other nodal heads in the system. The initial head in all other nodes can be selected in relation to the fixed head value.

The iteration starts in the internal cycle, where the nodal heads are determined by Equation 3.41. The calculation for each node is repeated until $H^{(k+1)} - H^{(k)} < \varepsilon_H$ is satisfied for all nodes in the system.

Thereafter, the flow will be calculated as $Q = \Delta H/U$ for each pipe, in the $(l+1)^{th}$ iteration of the external cycle. If $Q^{(l+1)} - Q^{(l)} > \varepsilon_Q$ for any of the pipes, the internal cycle will have to be repeated using the new U-values calculated from the latest flow rates.

The iteration stops when the requested flow accuracy has been achieved for all pipes or the specified maximum number of iterations was reached.

No matter how complicated the method looks at first glance, it is convenient for computer programming. Used for manual calculations, it will require lots of time even for a network of very few pipes. A spreadsheet application can reduce this but is by no means an alternative to a full-scale computer programme. It can however serve as a useful tool for better understanding of the principle.

Gradient method

The most recently used method to solve the network analysis problem, is the *Gradient method* that was first introduced by Todini and Pilati (1987). The linear system of equations formulated by their approach is solved using a matrix calculation described by George and Liu (1981). Next to its robustness, the additional advantage of this method is that it can handle the change of the status of system components (pumps and valves) without changing the structure of the equation matrix. The basic steps of the calculation procedure can be found in Rossman (2000).

Self-study:
Spreadsheet lesson A5.4.3 (Appendix 5)

3.6 PRESSURE-RELATED DEMAND

It is a known fact that more water is consumed when there is a higher pressure in the system, resulting in higher water flows at the outlets.

In addition to this, the leakage levels are very much pressure-sensitive, as can be seen from the British experience shown in Figure 3.21.

All calculation procedures explained in the previous sections deal with pipe flows against the hydraulic gradients, with the pressures calculated afterwards. As the reference head (pressure) is set independently from the flows, some error is introduced by neglecting the relation between the demand and pressure in the system. The mathematical relation between these two is quadratic, which can be derived from the analogy with the discharge through an orifice (see Figure 3.22).

The water pressure at the orifice is assumed to be atmospheric and applying the Bernoulli Equation for the cross-section just before and just after the orifice leads to the equation:

$$Q = CA\sqrt{2gh} \tag{3.42}$$

where A is the surface area of the orifice and C is a factor (< 1) related to its shape. With free surface level in the reservoir, water depth h above the orifice can be compared with energy head difference ($h \approx \Delta E \approx \Delta H$), while the C-factor corresponds to the minor loss factor,

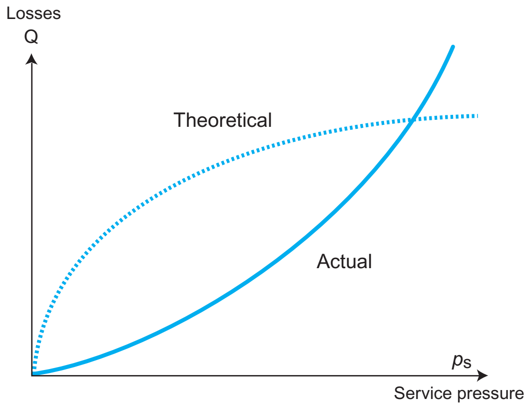


Figure 3.21. Pressure-related leakage (Wessex Water PLC, 1993).

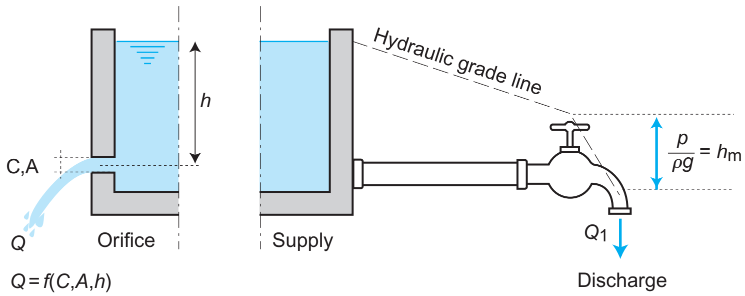


Figure 3.22. Discharge through an orifice.

ξ. Neglecting the friction, Equation 3.42 actually has a format comparable to $h_m = R_m Q^2$ shown in Equation 3.15 and further elaborated by Equation 3.25. Finally, it shows that the residual pressure in water distribution systems is destroyed at the tap, i.e. has in essence the status of a minor loss ($h_m = p/\rho g$).

In reality, applying this logic creates two potential problems:

1 Demand-driven hydraulic calculations will require a correction of the nodal discharges according to the calculated pressure, which may significantly increase the calculation time leading to an unstable iteration procedure. In other words, an input parameter (demand) becomes dependent on an output parameter (pressure).
2 Resistance R_m is, in the case of hundreds of nodes supplying thousands of consumers, virtually impossible to determine. In the best possible case, a general pressure-related diagram may be created from a series of field measurements.

From the Dutch experience, KIWA suggest a linear relation between the pressure and demand for calculations carried out for the assessment of distribution network reliability. The demand is considered independently of the pressure above a certain threshold, which is typically a pressure of 20–30 mwc (Figure 3.23).

Running hydraulic calculations for low-pressure conditions without taking the pressure-related demand into consideration might result in negative pressures in some nodes. This happens if for example the supply head is too low (see Figure 3.24) or the head-losses in the system are exceptionally high. Proper interpretation of the results is necessary in this case in order to avoid false conclusions about the system operation.

Applying the pressure-related demand mode in calculations causes a gradual reduction of the nodal discharges and the hydraulic gradient values, resulting in the slower drop of the reservoir levels, as Figure 3.25 shows.

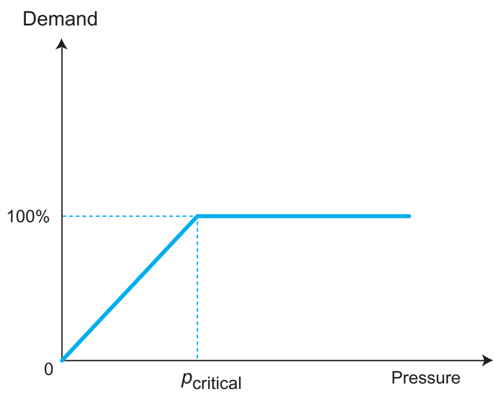


Figure 3.23. Pressure-related demand relation (KIWA, 1993).

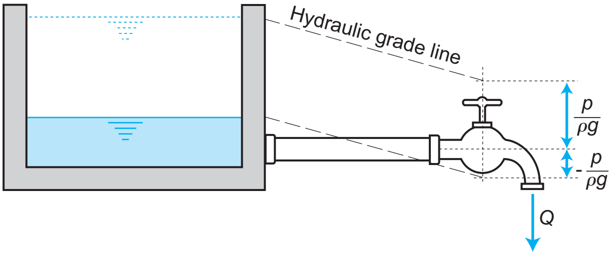


Figure 3.24. Negative pressures as a result of a calculation without pressure-related demand.

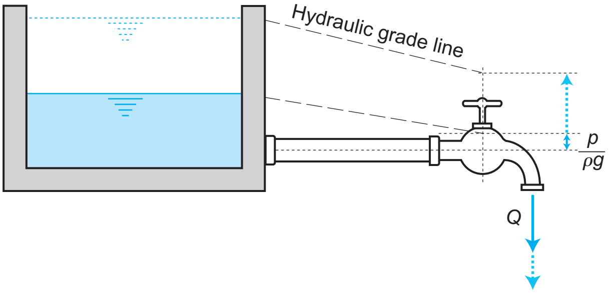


Figure 3.25. Pressures as the result of the calculation with pressure-related demand.

3.7 HYDRAULICS OF STORAGE AND PUMPS

Reservoirs and pumps are constructed to maintain the energy levels needed for water to reach the discharge points. Factors that directly influence the position, capacity and operation of these components are:
– topographical conditions,
– location of supply and demand points,
– patterns of demand variation,
– conveyance capacity of the network.

3.7.1 *System characteristics*

Pipe characteristics

The conveyance capacity of a pipe with a known length, diameter, roughness factor and slope is described by the *pipe characteristics* diagram. This diagram shows the required heads at the upstream side of the pipe, which enable the supply of a range of flows while maintaining constant pressure at the pipe end. The total head required for flow Q, in particular, consists of a dynamic and static component (see Figure 3.26).

Dynamic head

The *dynamic head* covers the head-losses i.e. the pipe resistance:

$$H_{dyn} = \Delta E = \Delta H = h_f + h_m \tag{3.43}$$

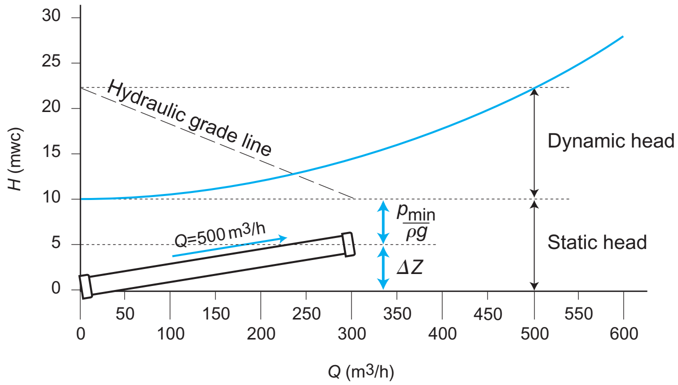


Figure 3.26. Pipe characteristics.

Static head

The *static head* is independent of the flow:

$$H_{st} = \frac{p_{end}}{\rho g} \pm \Delta Z \tag{3.44}$$

where p_{end} stands for the remaining pressure at the pipe end. In design problems where the required head is to be determined for the maximum flow expected in the pipe, the pressure at the end will be fixed at a critical value, i.e. $p_{end} = p_{min}$. *Maintaining the specified minimum pressure at any flow $Q \leq Q_{max}$ will result in the least energy input required for water conveyance at given pipe characteristics.*

ΔZ in Figure 3.26 is related to the pipe slope and represents the elevation difference between the end- and supplying point of the pipe. Positive ΔZ indicates the necessity of pumping while, if there is a negative value, the gravity may partly or entirely be involved in water conveyance. In the example from the figure, the static head $H_{st} = 10$ mwc comprises the minimum downstream pressure head of 5 mwc and 5 m of the elevation difference between the pipe ends. Such a pipe could deliver a maximum flow of 500 m^3/h if the head at the supply point was raised up to 22.5 mwc.

The curve for a single (transportation) pipe will be drawn for fixed L, D and k-values, and a range of flows covering the demand variations on a maximum consumption day. The minor losses are usually ignored and the curve will be plotted using the results of the friction loss calculation for various flow rates, as explained in Section 3.3.1.

System characteristics

In cases when the diagram in Figure 3.26 represents a network of pipes, it will be called the *system characteristics*. A quadratic relation between the system heads and flows can be assumed in this case:

$$H_{dyn} = R_s Q^2 \tag{3.45}$$

where R_s is the system resistance determined from pressure measurements for various demand scenarios.

The static head of the system will have the same meaning as in Formula 3.44 except that the end in this context means the most critical point i.e. with the lowest expected pressure. That point can exist at the physical end of the system, faraway from the source, but can also be within the system if located at a high elevation (thus, inflicting high ΔZ).

The system/pipe curves change their shape as the head-loss varies resulting from modification of the pipe roughness, diameter or length. This can be a consequence of:

– pipe ageing, and/or
– system rehabilitation/extension.

The curve becomes steeper in all cases as the head-loss increases, which results in the reduction of the conveying capacity. The original capacity can be restored by laying new pipes of a larger diameter, or in a parallel arrangement. Alternatively, more energy, i.e. the higher head at the supplying side, will be needed in order to meet the demand. The example in Figure 3.27 shows how the pipe from Figure 3.26 requires the upstream head of nearly 30 mwc after it loses its initial conveyance due to ageing, in order to keep the same supply of 500 m^3/h. Maintaining the initial supply head of 22.5 mwc would otherwise cause a reduction in supply to 400 m^3/h.

Self-study:
Spreadsheet lessons A5.1.6 and A5.2.1b (Appendix 5)

3.7.2 *Gravity systems*

In the case of gravity systems, the entire energy needed for water flow is provided from the elevation difference ΔZ. The pressure variation in the

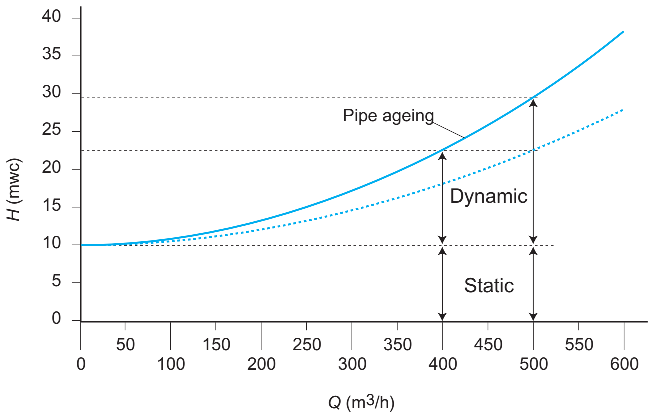


Figure 3.27. Capacity reduction of the system.

system is influenced exclusively by the demand variation (see Figure 3.28). Hence:

$$\Delta Z = H_{\text{dyn}} + H_{\text{st}} = \Delta H + \frac{P_{\text{end}}}{\rho g} \tag{3.46}$$

For known pressure at the end of the system, the maximum capacity can be determined from the system curve, as shown in Figure 3.29.

The figure shows that the lower demand overnight causes smaller head-loss and therefore the minimum pressure in the system will be higher than during the daytime when the demand and head-losses are higher. In theory, this has implications for the value of the static head that is changing with the variable minimum pressure. *The static head used for design purposes is always fixed based on the minimum pressure that is to be maintained in the system during the maximum consumption hour*.

When an area that is to be supplied from a single source starts to grow considerably, demand increases and longer pipe routes can lead to a pressure drop in the network affecting the newly-constructed areas. In

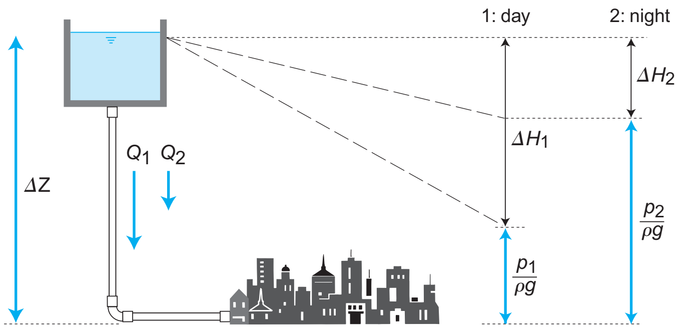


Figure 3.28. Gravity system: regular supply.

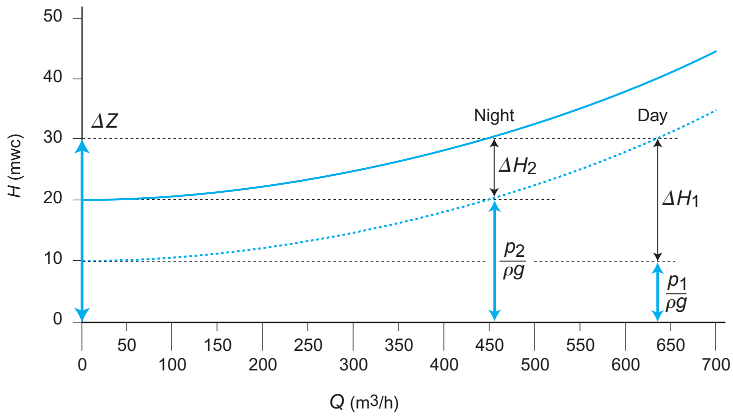


Figure 3.29. System characteristics: regular operation.

theory, this problem can be solved by enlarging the pipes and/or elevating the reservoir (Figures 3.30 and 3.31). Nonetheless, the latter is often impossible due to the fixed position of the source and additional head will probably have to be provided by pumping.

Zero-line

If the system is supplied from more than one side, the storage that is at the higher elevation will normally provide more water i.e. the coverage of the larger part of the distribution area. The intersection between the hydraulic grade lines shows the line of separation between the areas covered by different reservoirs, the so-called *zero-line* (Figure 3.32).

Hydraulic conditions in the vicinity of the zero-line are unfavourable:

– the pressure is lower than in other parts of the network,
– the flow velocities are also low, leading to water stagnation and potential water quality problems.

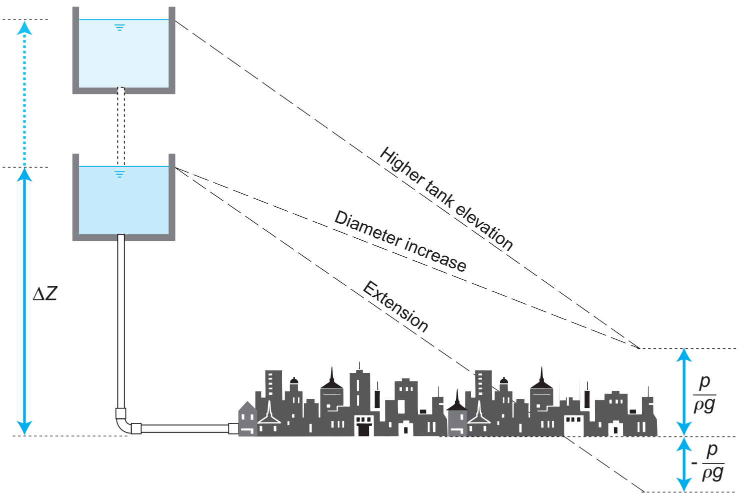


Figure 3.30. Gravity system: network extension.

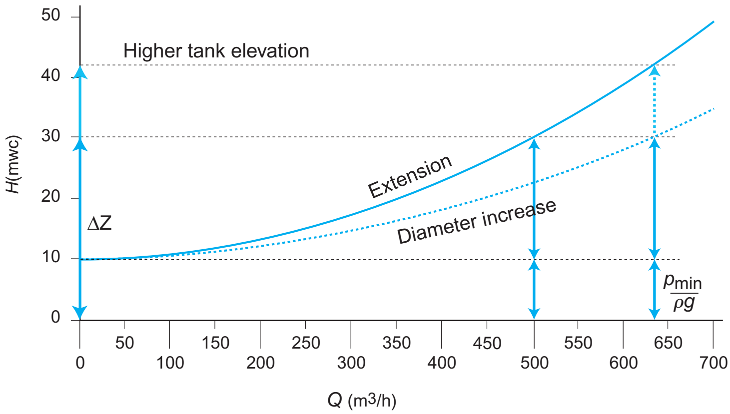


Figure 3.31. System characteristics: network extension.

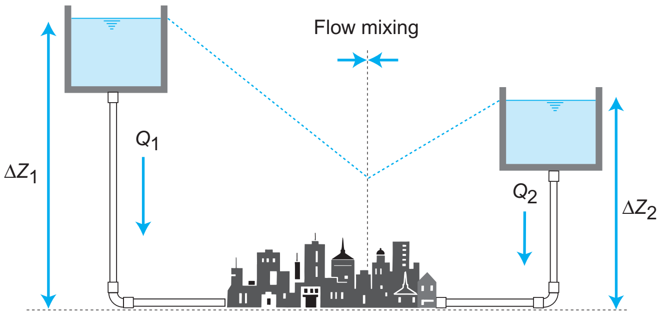
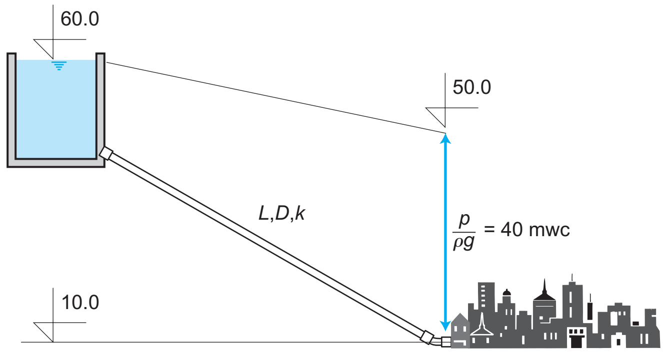


Figure 3.32. Gravity system: supply from two sides.

PROBLEM 3.13

For the gravity system shown in the figure, find the diameter of the pipe $L = 2000$ m that can deliver a flow of 6000 m^3/h with a pressure of 40 mwc at the entrance of the city. Absolute roughness of the pipe can be assumed at $k = 1$ mm and the water temperature equals 10°C.

What will the increase in capacity of the system be if the pressure at the entrance of the city drops to 30 mwc?



Answers:

At the elevation of $Z = 10$ msl and the pressure of $p/\rho g = 40$ mwc, the piezometric head at the entrance of the city becomes $H = 50$ msl. The surface level/piezometric head of the reservoir is 10 metres higher, and this difference can be utilised as friction loss. The hydraulic gradient then becomes $S = h_f/L = 10/2000 = 0.005$. From the hydraulic tables in Appendix 4, for $k = 1$ mm and $T = 10$°C:

Discharge flows (l/s), $k = 1$ mm, S = 0.005

D (mm)	Q (m^3/h)
900	4772.0
1000	6292.6

If the pressure drops to $p/\rho g = 30$ mwc, the available friction loss increases to $h_f = 20$ mwc and therefore $S = 0.01$. From the same tables, for $D = 1000$ mm, the flow that can be supplied for the given hydraulic gradient increases to $Q = 8911.1$ m^3/h.

The same results can be obtained by the calculation procedures demonstrated in Problems 3.8 and 3.7 in Sections 3.3.3 and 3.3.2, respectively.

Self-study:
Workshop problem A1.5.1 (Appendix 1)
Spreadsheet lessons A5.5.1–A5.5.5 (Appendix 5)

3.7.3 *Pumped systems*

Pump lift

Suction and discharge pipe

In pumped systems, the energy needed for water conveyance is obtained from the pump operation. This energy, generated by the pump impeller, is usually expressed as a head of water column (in mwc) and is called the *pumping head* (or *pump lift*), h_p. It represents the difference between the energy levels at the pump entrance i.e. at the *suction pipe* and at the pump exit, i.e. at the *discharge (*or *pressure) pipe* (Figure 3.33).

In the case of a single pump unit, the higher the pumping head h_p is, the smaller the pumped flow Q will be. For a combination of $Q-h_p$ values, the power N (W) required to lift the water is calculated as:

$$N = \rho g Q h_p \tag{3.47}$$

Pump discharge

where Q (m^3/s) is the *pump discharge*. The power to drive the pump will be higher, due to energy losses in the pump:

$$N_p = \frac{\rho g Q h_p}{\eta_p} \tag{3.48}$$

where η_p is the pump efficiency dependant on the pump model and working regime. Finally, the power required for the pump motor will be:

$$N_m = \frac{N_p}{\eta_m} \tag{3.49}$$

where η_m indicates the motor efficiency.

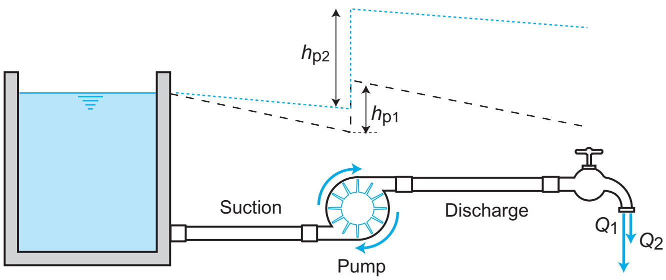


Figure 3.33. Pumping head.

Pump characteristics

The hydraulic performance of pumps is described by the *pump characteristics*. This diagram shows the relation between the pump discharge and delivered head (Figure 3.34).

For centrifugal pumps, a very good approximation of the pump curve is achieved by the following equation:

$$h_p = aQ^2 + bQ + c \tag{3.50}$$

where factors a, b and c depend on the pump model and flow units. The alternative equation can also be used:

$$h_p = c - aQ^b \tag{3.51}$$

Duty flow and head

This equation allows for the pump curve definition with a single set of Q-h_p points (Rossman, 2000). These are known as the duty flow (Q_d) and duty head (H_d) and indicate the optimal operational regime of the pump, i.e. the one in which the maximum efficiency η_p will be achieved. As a convention, for exponent $b = 2$:

$$h_{p(Q=0)} = c = \frac{4}{3}H_d\,; \quad Q_{(h_p=0)} = 2Q_d \Rightarrow a = \frac{1}{3}\frac{H_d}{Q_d^2} \tag{3.52}$$

Pump manufacturers regularly supply pump characteristics diagrams for each model; a typical format showing a range of impeller diameters and efficiencies η_p, is given in Figure 3.35.

Following the discussions in Section 3.7.1, the pumping head required at the supply side of the system to maintain certain minimum pressure at its end will be:

$$h_p^{req} = H_{dyn} + H_{st} = \Delta H + \frac{P_{min}}{\rho g} \pm \Delta Z \tag{3.53}$$

This required head is normally higher during daytime than overnight, resulting from higher demand i.e. higher head-losses. Operating the

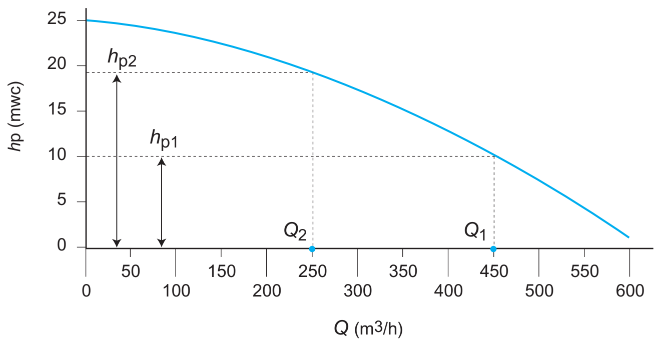


Figure 3.34. Pumping characteristics.

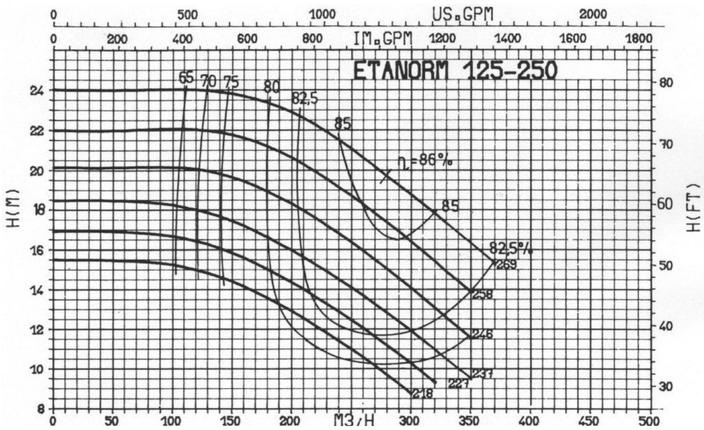


Figure 3.35. Pump characteristics from a manufacturer's catalogue (KSB, 1990).

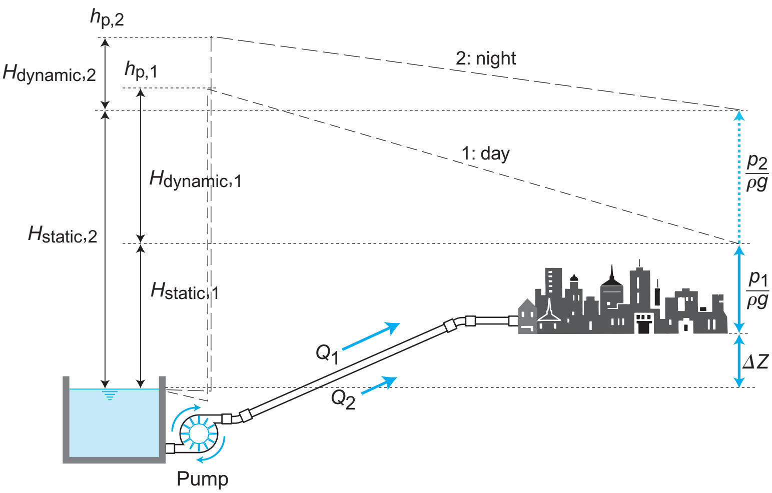


Figure 3.36. Operation of one pump.

same pump (curve) over 24 hours is therefore unfavourable as it results in the opposite effect: low heads during the daytime and high heads overnight (Figure 3.36). Apart from that, using a single pump in a pumping station is unjustified for reasons of low reliability, high-energy consumption/low efficiency and problematic maintenance. In practice, several pumps are commonly combined in one pumping station.

Working point

Flows and pressures that can be delivered by pump operation are determined from the system and pump characteristics. The intersection of these two curves, the *working point*, indicates the required pumping head that provides the flow and the static head, as shown on the graph in Figure 3.37.

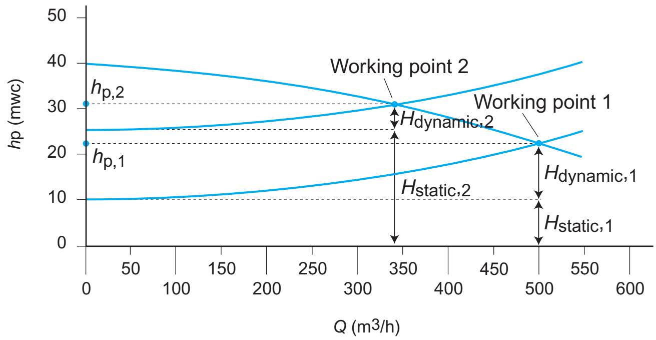


Figure 3.37. Operation of one pump: day- and night time flows.

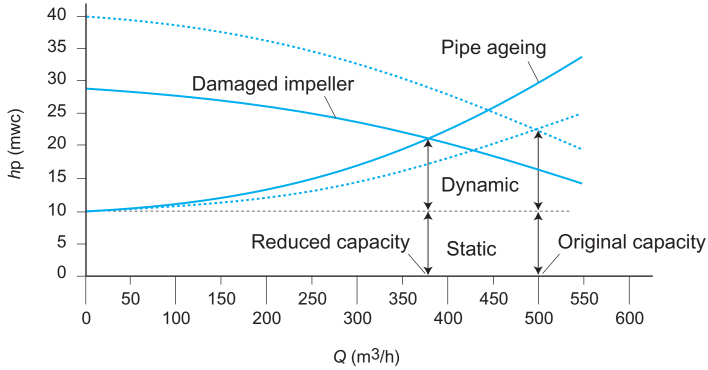


Figure 3.38. Operation of one pump: flow reduction.

As in the case of the gravity supply discussed in the previous section, the pressure variation in the distribution area has implications for the value of the static head in this case as well. For design purposes, however, *the working point obtained from the system characteristics plotted at the lowest static head (i.e. the lowest pressure required in the system) will be used to determine the maximum pump capacity*.

The maximum pumping capacity may vary in time. Ageing of the pump impeller, pipe corrosion, increase of leakage, etc. will cause reduction of the maximum flow that can be delivered by the pump while maintaining the same static head (Figure 3.38).

Decisions on the number and size of pumps in a pumping station are made with the general intention of keeping the pressure variations in the system at the lowest acceptable level in order to minimise the required pumping energy. For this reason, several pumps connected to the same delivery main can be installed in parallel. Their operation will be represented by a composite pump curve, which is obtained by adding the single pump discharges at the same pumping head. Hence, for n pumps:

$$h_p = h_1 = h_2 = h_3 = \cdots = h_n$$

$$Q_p = Q_1 + Q_2 + Q_3 + \cdots + Q_n$$

Figure 3.39 shows the operation of two equal pumping units in parallel arrangement. The system should preferably operate at any point along the curve A_1-A_2-B_1-B_2, between the minimum and maximum flow.

The shaded area on the figure indicates excessive pumping, which is unavoidable when fixed speed pumps are used. A properly-selected combination of pump units should reduce this area. This is often achieved by installing pumps of different capacities; the example in Figure 3.40 shows the combination of three equal units with one stronger pump.

Introducing variable speed pumps can completely eliminate the excessive head. The flow variation is in this case met by adjusting the impeller rotation, keeping the discharge pressure constant (Figure 3.41).

The pump characteristics diagram will consist of a family of curves for various pump frequencies, *n* (rpm). The relation between the various pumping heads and flows of any two curves is proportional to the frequencies in the following way:

$$\frac{Q_2}{Q_1} = \frac{n_2}{n_1}; \quad \frac{h_{p,2}}{h_{p,1}} = \left(\frac{n_2}{n_1}\right)^2 \tag{3.54}$$

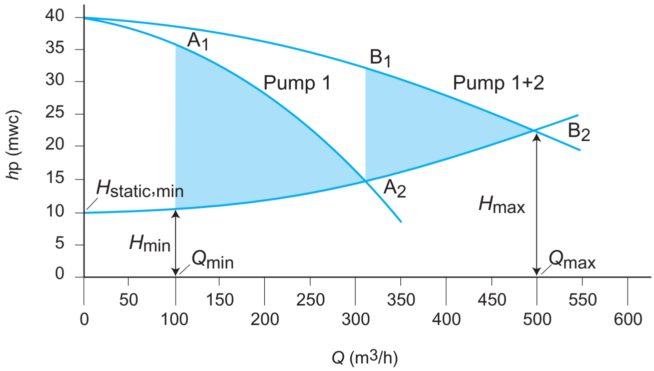


Figure 3.39. Equal pumps in parallel arrangement.

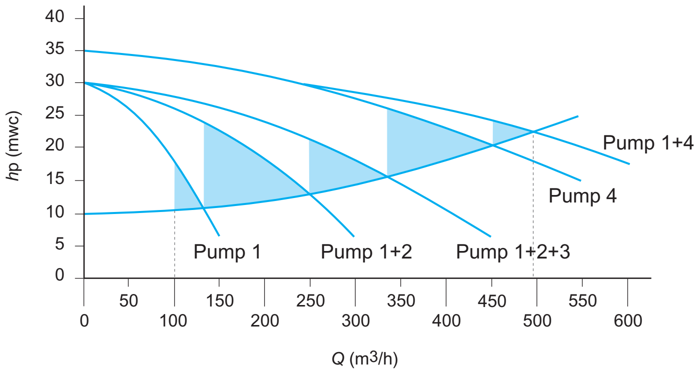


Figure 3.40. Various pump sizes in a parallel arrangement.

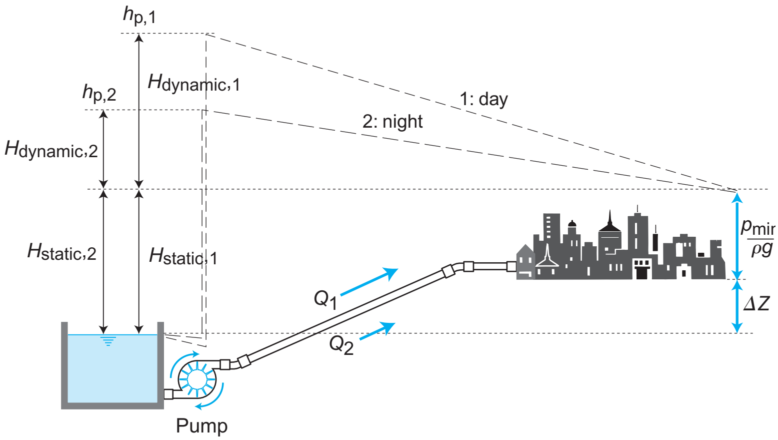


Figure 3.41. Operation of variable speed pumps.

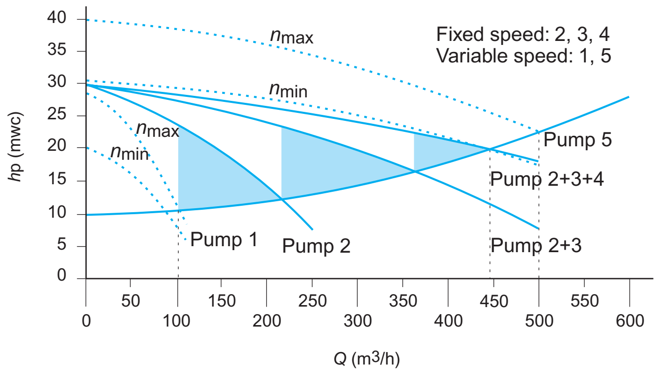


Figure 3.42. Combined operation of variable and fixed speed pumps.

Nevertheless, one variable speed unit alone can hardly cover the entire range of flows and several units in parallel will therefore be used. Variable speed pumps can also be combined with fixed speed pumps, controlling only the peak flows (Figure 3.42).

In case of large pressure variations in the system, the pumps have to be installed in a serial arrangement. The total head is in this case equal to the sum of heads for each pump. Figure 3.43 shows the curves for two pumps in operation. For n equal units:

$$h_p = h_1 + h_2 + h_3 + \cdots h_n$$

$$Q_p = Q_1 = Q_2 = Q_3 = \cdots Q_n$$

In addition to the discussion at the end of Section 3.7.2, pumping from more than one supply point will cause similar problems as with gravity systems in areas where the water from different sources is mixed. However, modifying the pump regimes can shift the zones of minimum pressure, as shown in Figure 3.44.

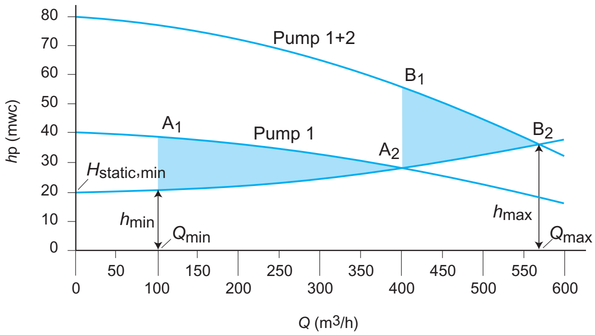


Figure 3.43. Pumps in series.

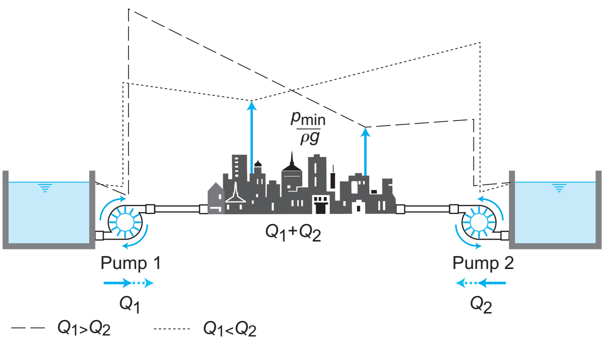
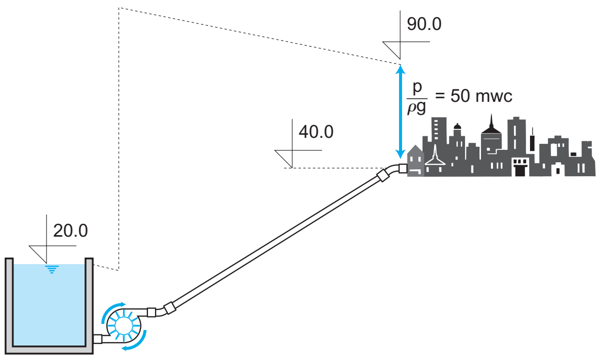


Figure 3.44. Pumping from two sources.

PROBLEM 3.14

For the pumped system shown in the following figure, determine the required pumping head to deliver flow $Q = 4000\ \text{m}^3/\text{h}$ through pipe $L = 1200$ m and $D = 800$ mm, while maintaining the pressure of 50 mwc at the entrance of the city. Absolute roughness of the pipe can be assumed at $k = 0.5$ mm and the water temperature equals 10° C.

Find the equation of the pump curve using Equations 3.51 and 3.52 assuming the earlier operation happens at maximum pump efficiency. What will be the pressure at the entrance of the city when there is a demand growth of 25%?



Answers:

At the entrance of the city, elevation $Z = 40$ msl. With a required pressure of $p/\rho g = 50$ mwc, the piezometric head becomes $H = 90$ msl. As the surface level/piezometric head of the reservoir is set at 20 metres, the total static head $H_{st} = 90–20 = 70$ mwc. The losses between the reservoir and the pump can be ignored.

For the following parameters: $Q = 4000\ m^3/h$, $L = 1200$ m, $D = 800$ mm, $k = 0.5$ mm and $T = 10°C$, the pipe friction loss will be calculated as follows:

$$v = \frac{4Q}{D^2\pi} = \frac{4\times4000}{0.8^2\times3.14\times3600} = 2.21\ \text{m/s}$$

For temperature $T = 10\ °\ C$, the kinematic viscosity from Equation 3.22, $\upsilon = 1.31\times10^{-6}\ m^2/s$. The Reynolds number takes the value of:

$$\text{Re} = \frac{vD}{\upsilon} = \frac{2.21\times0.8}{1.31\times10^{-6}} = 1.4\times10^6$$

and the friction factor λ from Barr's Equation equals:

$$\lambda = 0.25\Bigg/\log^2\left[\frac{5.1286}{Re^{0.89}} + \frac{k}{3.7D}\right]$$

$$= 0.25\Bigg/\log^2\left[\frac{5.1286}{(1.4\times10^6)^{0.89}} + \frac{0.5}{3.7\times800}\right] \approx 0.018$$

The friction loss from the Darcy–Weisbach Equation can be determined as:

$$h_f = \frac{\lambda L}{12.1D^5}Q^2 = \frac{0.018\times1200}{12.1\times0.8^5}1.11^2 \approx 7\ \text{mwc}$$

The total required pumping head is therefore $h_p = H_{st} + H_{dyn} = 70 + 7 = 77$ mwc. Given the maximum pumping efficiency, this is also the duty head at the duty flow of 4000 m^3/h and in Equation 3.51:

$$a = \frac{1}{3}\frac{H_d}{Q_d^2} = \frac{77}{3\times4000^2} = 1.604\times10^{-6}\ \text{and}$$

$$c = \frac{4}{3}H_d = \frac{4}{3}\times77 = 102.67$$

Hence, the pumping curve can be approximated with the following equation (exponent $b = 2$):

$$h_p = c - aQ^b \approx 103 - 1.6\times10^{-6}Q^2$$

If demand grows by 25% i.e. to 5000 m^3/h, the pumping head that can be provided will be:

$$h_p = 103 - 1.6\times10^{-6}5000^2 \approx 63 \text{ mwc}$$

The friction loss calculated in the same way as above is going to increase to approximately 11 mwc, leading to a residual pressure at the entrance to the city of $p/\rho g = 20 + 63 - 11 - 40 \approx 32$ mwc.

Self-study:
Workshop problem A5.2 (Appendix 1)
Spreadsheet lessons A5.6.1–A5.6.5 (Appendix 5)

3.7.4 *Combined systems*

Consumers in combined systems are partly supplied by gravity and partly by pumping. Three basic concepts can be distinguished:

1 The water is pumped from a reservoir into the distribution area (tank-pump-network).
2 The water is pumped to a reservoir and thereafter supplied by gravity (pump-tank-network).
3 Pump and reservoir are at the opposite sides of the distribution area (pump-network-tank).

Tank-pump-network

This scheme is suitable for mild terrains where a favourable location for the reservoir is difficult to find, either due to insufficiently-high elevations or because of a large distance from the distribution area.

Essentially this is the same concept as that of direct pumping, except that the required pumping head can be reduced on account of the elevation difference in the system. Hence:

$$h_p + \Delta Z = H_{dyn} + H_{st} = \Delta H + \frac{P_{end}}{\rho g} \tag{3.55}$$

Both the dynamic and static head are supplied partly by gravity and partly by pumping, depending on the elevation difference and the pressure at the end of the system (Figures 3.45 and 3.46).

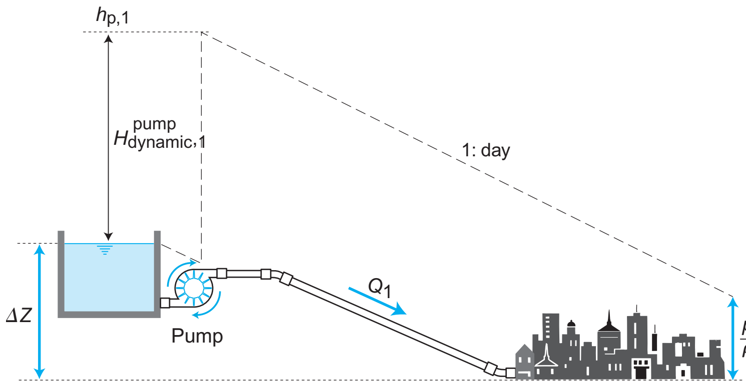


Figure 3.45. Combined supply by gravity and pumping: daytime flows.

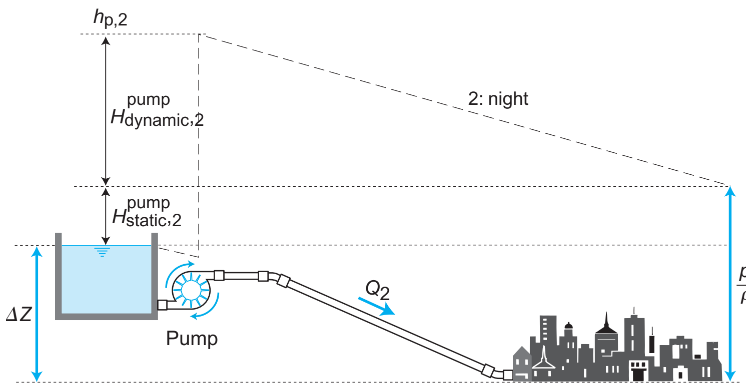


Figure 3.46. Combined supply by gravity and pumping: night time flows.

Booster station

Pumping stations need not necessarily to be located at the supply point. When positioned within the system, they are commonly called *booster stations*. Such a layout is attractive if high pressures are to be avoided (Figure 3.47).

Pump-tank-network
This scheme is typical for hilly terrains. When pumps deliver water to the reservoir, the static head will only comprise the elevation head ΔZ, which equals the elevation difference between the surface levels in the two reservoirs (Figure 3.48). Thus:

$$h_p = H_{dyn} + H_{st} = \Delta H + \Delta Z \tag{3.54}$$

The advantages of this scheme are:
– stable operation of the pumping station,
– a buffer supply capacity in case of pump failure.

A similar hydraulic pattern is valid if water towers are put into the system (Figure 3.49). However, their predominant role is to maintain stable

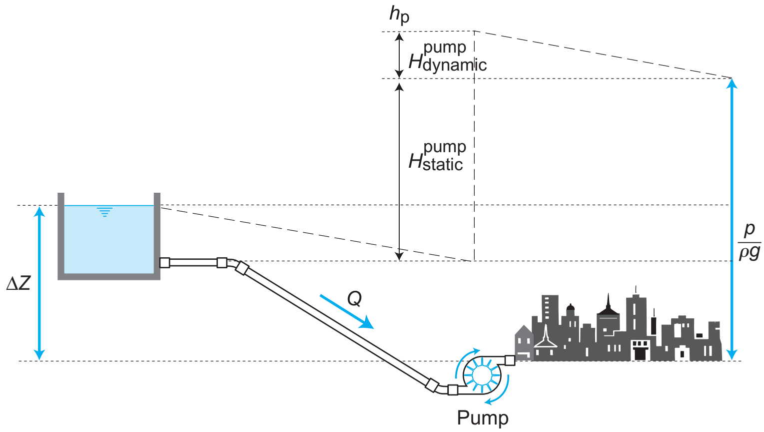


Figure 3.47. Booster stations.

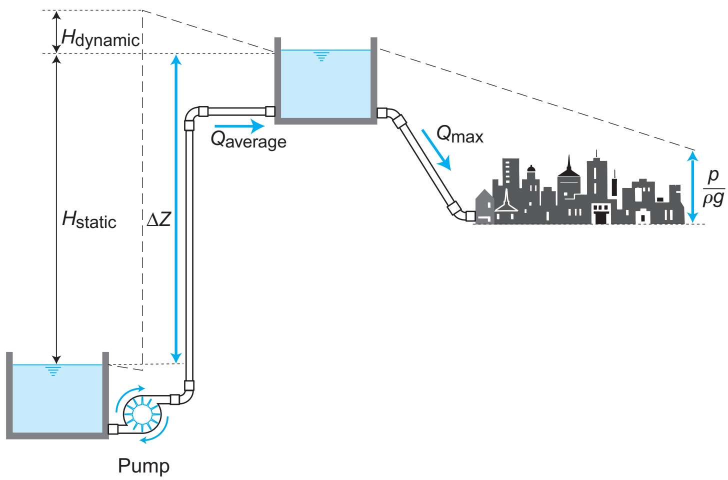


Figure 3.48. Gravity supply supported by pumping.

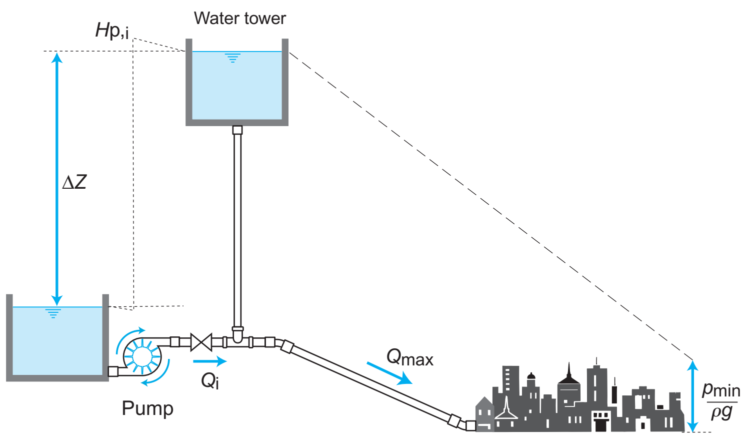


Figure 3.49. Pump operation in combination with water tower.

operation of the pumps, rather than to provide buffer- or large balancing volumes.

While supplying tanks, the pumps often operate automatically, based on monitoring of water levels in the reservoirs. Pump throttling may be required in order to adjust the flow. The effects on the system characteristics are shown in Figure 3.50.

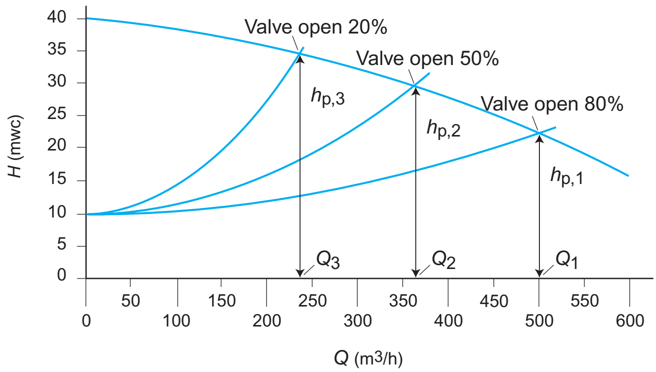


Figure 3.50. Effects of pump throttling on system characteristics.

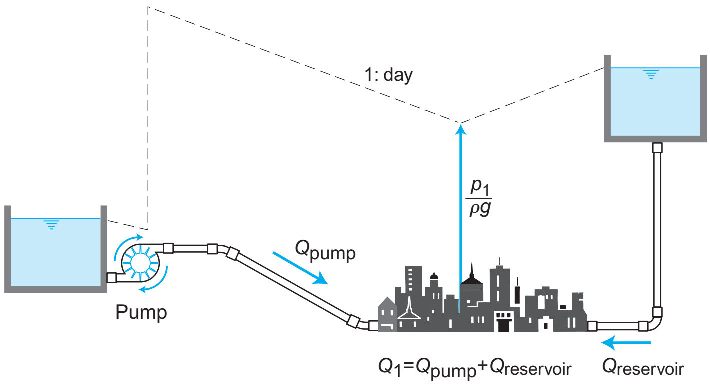


Figure 3.51. Counter tank: daytime flows.

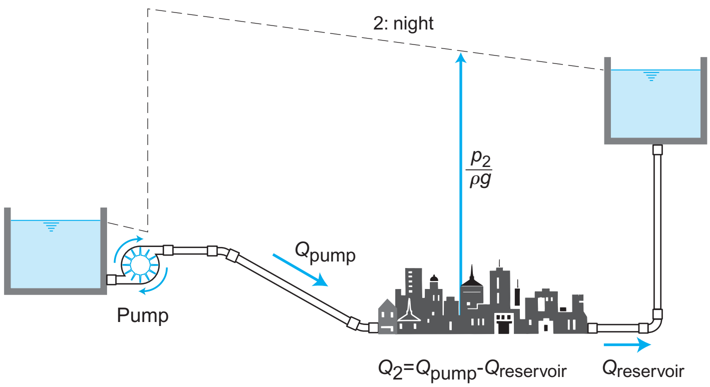


Figure 3.52. Counter tank: night time flows.

Pump-network-tank
This scheme is predominantly applied for distribution networks located in valleys. During the maximum supply conditions, both the pump and reservoir will cover part of the distribution area (Figure 3.51). If the only source of supply is close to the pumping station, that one will also be used to refill the volume of the tank. This is usually done overnight when the demand in the area is low (Figure 3.52).

Counter tank

Tank operating in this way functions as a kind of *counter tank* to the one at the source. Depending on its size and elevation, it can balance the demand variation in the system, partly or completely. In the second case, the pumping station operates at constant (average) capacity ($Q_{pump} = Q_{average}$).

Self-study:
Workshop problems A1.5.3–A1.5.5 (Appendix 1)
Spreadsheet lessons A5.7.1–A5.7.4 (Appendix 5)

CHAPTER 4

The Design of Water Transport and Distribution Systems

The design of water transport and distribution systems consists of two parts: hydraulic and engineering. The main parameters considered in hydraulic design have been discussed in previous chapters. Apart from sufficient flows, pressures and velocities, a well-designed system should fulfil the following additional requirements:

– minimised operational costs in regular supply conditions,
– reasonable supply during irregular situations (power/pump failure, pipe burst, fire events, system maintenance, rehabilitation or reconstruction) and
– flexibility with respect to future extensions.

Engineering design criteria

Keeping the hydraulic parameters within an acceptable range cannot by itself fulfil these requirements. Equally important are so-called *engineering (non-hydraulic)* design criteria, such as:

– the selection of durable pipe materials, joints, fittings and other appurtenances,
– setting a network of valves whereby parts of the network can quickly be isolated and
– providing easy access to the vital parts of the system, etc.

Respecting both the hydraulic and engineering design criteria guarantees satisfactory operation of the system throughout the entire design period.

4.1 THE PLANNING PHASE

Choosing to commission a water distribution system means a huge investment with far-reaching implications for the development of the area that will be covered by the network. To avoid major mistakes, starting with a good plan is a meaningful preparatory step before the detailed design considerations take place. The planning phase has to answer the following questions (Pieterse, 1991):

1 Is the project feasible?
2 What is the best global approach?

3 What are the estimated costs?
4 What is the required timescale for execution?

Looking for appropriate answers in this case is often a complex assignment in which experts of different profiles are involved. Hence, organizing the work effectively is an essential element of the planning. The job normally starts by establishing a project management team with the following main tasks:
– a project review,
– a survey of required expertise and equipment,
– the securing of cooperation between involved organisations,
– the setting of project objectives with respect to time, costs and quality.

Before thinking about any possible solution, existing information and ideas about the long-term physical planning objectives of the distribution system are to be explored. The main strategy of the long-term development of the region is usually stipulated in documents prepared at governmental level. Based on these plans, more specific analyses related to the aspects of water supply will lead to a number of concept solutions. These alternatives are discussed and evaluated by the studies that form the actual essence of the design (identification report, feasibility study, master plan). Apart from global recommendations on how to approach the design, the outcome of these studies will result in the more detailed organization of the project, such as:
– division of the project into smaller parts,
– definition of project phases (in terms of time),
– estimates of costs and time necessary for the execution.

Approving these steps and organising successful fund-raising are preconditions for starting of the design phase.

Conclusions are always made with a margin in the planning phase. This is logical, as a period of 20 to 30 years is long enough to include unforeseen events arising from political problems, natural disasters, epidemics, and other (not always negative) factors distorting normal population growth. It is therefore wise to develop water distribution facilities in stages, following the actual development of the area. This principle allows the gradual accumulation of funds for investment, as well as the intermediate evaluation and adaptation of the design where actual development deviates from the original planning. Thus, the planning phase is never fully completed before the design and execution phases begin.

4.1.1 *The design period*

Design period

Various components of the distribution system are designed for a certain period of time called the *design period*. During this period, the capacity

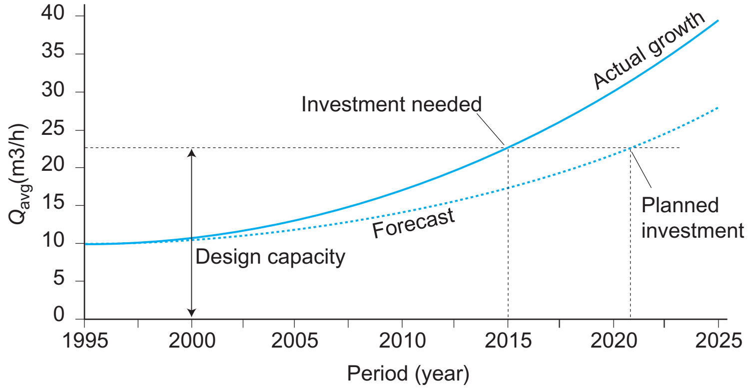


Figure 4.1. Demand forecast.

of the component should be adequate unless the actual water demand differs from the forecast, as Figure 4.1 shows.

Technical lifetime

The *technical lifetime* of a system component represents the period during which it operates satisfactorily in a technical sense. The suggested periods for the main distribution system components shown in Table 4.1 indicate a wide range that mostly depends on appropriateness of the choice and the way in which the component has been maintained.

Economic lifetime

The *economic lifetime* represents the period of time for which the component can operate before it becomes more costly than its replacement. This lifetime is never longer than the technical lifetime; very often it is much shorter. Its estimation is complex and depends on aspects such as operation and maintenance costs, technological advancement and interest rates.

In practice, the design period is often the same as the economic lifetime. Moreover, a uniform design period will be chosen for all components; design periods of 20–25 years are typical for distribution systems. An exception is mechanical equipment in pumping stations, which has a lifetime of 10–15 years. Although water companies are sometimes able to successfully maintain pumps operating for longer than 30 years, or

Table 4.1. Technical lifetime of distribution system components.

Component	Period (years)
Transmission mains	30–60
Distribution mains	30–80
Reservoirs	20–80
Pumping station – facilities	20–80
Pumping station – equipment	15–40

pipes with low corrosion that are older than 70 years, experience shows that design periods rarely exceed 30 years. Design periods shorter than 10 years are uneconomic and therefore undesirable.

4.1.2 *Economic aspects*

The economic comparison of design alternatives is a key element of the final choice; at the same time this is the most debatable part of the whole project.

For practical reasons, the alternatives will be compared within the same design period for all components, although the most economic design period may differ for individual components. The important factors that influence the most economic design period are:
– interest rates,
– inflation rates,
– energy prices,
– water demand growth,
– the 'scale' economy.

The 'scale' economy is an approach where investment costs are established in relation to the main properties of the system component. This is possible if the water supply company, or a number of neighbouring companies, have kept sufficient records of relevant costs.

First cost

For instance, the *first cost* (FC) of concrete reservoirs can be calculated as $a \times V^n$, where V is the tank volume in m^3, and a and n the factors depending on local conditions. A similar relation can be used for pumping stations taking the maximum capacity Q instead of volume V into consideration. Furthermore, linear or exponential relations can be adopted for transmission lines as, for instance, FC $= a \times D$, or FC $= b + c \times D^n$, with D representing the pipe diameter, say in millimetres.

Present/Annual worth

A preliminary cost comparison of the considered design alternatives can be carried out using the *present worth* (present value) or the *annual worth* method.

Single present worth factor

By the present worth method, all actual and future investments are calculated back to a reference year, which in general is the year of the first investment. The alternative with the lowest present value offers the most economic solution. The basic parameter in the calculation is the *single present worth factor*, $p_{n/r}$:

$$p_{n/r} = \frac{1}{s_{n,r}} = \frac{1}{(1 + r)^n} \tag{4.1}$$

Single compound amount factor

where $s_{n/r}$ is the *single compound amount factor*, which represents the growth of the present worth PW after *n* years with a *compounded interest rate* of *r*. The present worth of the future sum F then becomes $PW = F \times p_{n/r}$.

According to the annual worth method, a present principal sum P is equivalent to a series of n end-of-period sums A, where:

$$A = P\frac{r(1+r)^n}{(1+r)^n - 1} = P \times a_{n/r} \tag{4.2}$$

Annuity

In Equation 4.2, $a_{n/r}$ represents the *capital recovery factor* (*annuity*) When the present worth is calculated as $PW = A/a_{n/r}$ the $1/a_{n/r}$ is called the *uniform present worth factor*.

Annual inflation rate

Use of an *ideal interest rate i* in Equations 4.1 and 4.2, instead of the true interest rate *r*, allows the impact of inflation to be taken into account. Factor *f* in Equation 4.3 represents the *annual inflation rate*.

$$i = \frac{r - f}{1 + f} \tag{4.3}$$

The Theory of Engineering Economy offers more sophisticated cost evaluations that can be further studied in appropriate literature; for further information refer for instance to De Garmo *et al*. (1993).

The most economic alternative usually becomes obvious after comparisons between the investment and operational costs and their effects on the hydraulic performance of the component/system. A simplified principle to evaluate the investment- and operation and maintenance (O&M) costs for a trunk main is demonstrated further in this paragraph.

A pipe conveys flow Q (in m^3/s) while generating head-loss ΔH (mwc). The cost of energy EC (kWh) wasted over time T (hours) can be calculated as:

$$EC = \frac{\rho g Q \Delta H}{1000 \times \eta} T \times e \tag{4.4}$$

where e is the unit price (per kWh) of the energy needed to compensate the pipe head-loss. By supplying this energy by a pump, the annual costs of the energy wasted per metre length of the pipe become:

$$EC = \frac{9.81 \times 24 \times 365 \times Q\Delta H}{3600 \times L}\frac{e}{\eta} \approx 24 \times Q\frac{e}{\eta}\frac{\Delta H}{L} \tag{4.5}$$

where Q is the average pump flow in m^3/h, and η is the corresponding pumping efficiency. Substituting the hydraulic gradient, $\Delta H/L$ by using the Darcy–Weisbach Equation, the energy cost per annum will be (assuming the friction factor λ is equal to 0.02):

$$EC = 24 \times Q \frac{e}{\eta} \frac{0.02 \times Q^2}{12.1 \times D^5 \times 3600^2} \approx 3 \times 10^{-9} \frac{e}{\eta} \frac{Q^3}{D^5} \tag{4.6}$$

where D is the pipe diameter expressed in metres (and Q in m^3/h). By adopting a linear proportion between the pipe diameter and its cost, the total annual costs including investment and operation of the pipe are:

$$A = a \times D \times a_{n/r} + 3 \times 10^{-9} \frac{e}{\eta} \frac{Q^3}{D^5} \tag{4.7}$$

Equation 4.7 has the optimum solution if $\delta A/\delta D = 0$:

$$a \times a_{n/r} = 5 \times 3 \times 10^{-9} \frac{e}{\eta} \frac{Q^3}{D^6} \tag{4.8}$$

which finally results in the most economic diameter:

$$D = 0.05\sqrt{Q} \sqrt[6]{\frac{e}{\eta a_{n/r} a}} \tag{4.9}$$

Equation 4.9 considers fixed energy costs and water demand over the design period. The growth of these parameters should also normally be taken into account.

Essentially Figure 4.2 has the same approach. The diagram in this Figure shows investment and operational costs calculated for a range of possible diameters. The larger diameters will obviously be more expensive while generating lower friction losses i.e. generating the lower energy costs. The minimum of the curve summarising these two costs pinpoints the most economic diameter, in this case of 300 mm.

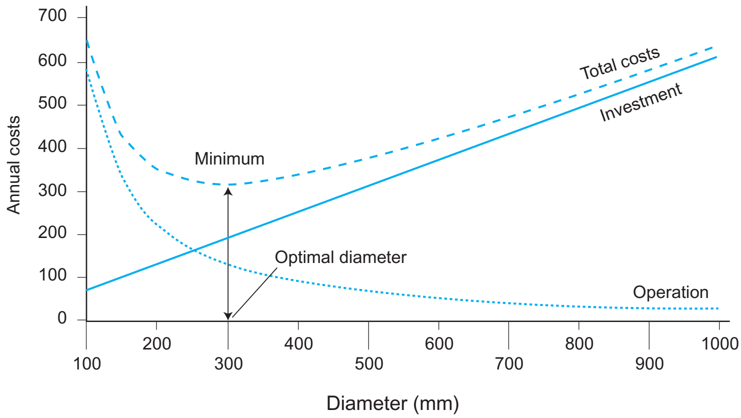


Figure 4.2. Costs comparison of the optimum diameter.

PROBLEM 4.1

A loan of US$ 5,000,000 has been obtained for reconstruction of a water distribution system. The loan has an interest rate of 6% and repayment period of 30 years. According to alternative A, 40% of this loan will be invested in the first year and 30% in years two and three, respectively. Alternative B proposes 60% of the loan to be invested in year one and the rest in year 10. Which of the two alternatives is cheaper in terms of investment? Calculate the annual instalments if the repayment of the loan starts immediately. What will be the situation if the repayment of the loan starts after 10 years?

Answers:

The present worth for both alternatives will be calculated for the beginning of the period. In alternative A:

$$PW_A = \sum_{i=1}^{3} F_i p_{in/6} = 5{,}000{,}000 \times \left[\frac{0.4}{(1+0.06)^1} + \frac{0.3}{(1+0.06)^2} + \frac{0.3}{(1+0.06)^3}\right] = 4{,}481{,}216 \text{ US\$}$$

while for alternative B:

$$PW_B = \sum_{i=1}^{2} F_i p_{in/6} = 5{,}000{,}000 \times \left[\frac{0.6}{(1+0.06)^1} + \frac{0.4}{(1+0.06)^{10}}\right] = 3{,}946{,}978 \text{ US\$}$$

Due to the postponed investments, alternative B appears to be more cost effective. The annuity calculated from Equation 4.2 for a repayment period of 30 years and interest rate of 6% becomes:

$$a_{30/6} = \frac{0.06 \times (1+0.06)^{30}}{(1+0.06)^{30} - 1} = 0.0726$$

leading to 30 annual instalments of $0.0726 \times 4{,}481{,}216 = 325{,}336$ US$ in case of alternative A, and $0.0726 \times 3{,}946{,}978 = 286{,}550$ US$ for alternative B.

If the repayment of the loan is delayed for 10 years i.e. stretches over 20 years, the calculated annuity becomes:

$$a_{20/6} = \frac{0.06 \times (1+0.06)^{20}}{(1+0.06)^{20} - 1} = 0.0872$$

For the same schedule of investments, the present value in year 10 in alternative A becomes:

$$PW_{A,10} = 5{,}000{,}000 \times \left[\frac{0.4}{(1+0.06)^{-9}} + \frac{0.3}{(1+0.06)^{-8}} + \frac{0.3}{(1+0.06)^{-7}}\right] = 8{,}025{,}175 \text{ US\$}$$

while in alternative B:

$$PW_{B,10} = 5{,}000{,}000 \times \left[\frac{0.6}{(1+0.06)^{-9}} + \frac{0.4}{(1+0.06)^{0}}\right] = 7{,}068{,}437 \text{ US\$}$$

The annual repayments starting from this moment will be 0.0872 × 8,025,175 = 699,795 US$ in alternative A, and 0.0872 × 7,068,437 = 616,368 US$ for alternative B. These are to be paid for a period of 20 years.

PROBLEM 4.2
Calculate the most economic diameter of the transmission line that transports an average flow $Q = 400$ m^3/h. The price of energy can be assumed at 0.15 US$ per kWh and the average pumping efficiency is 65%. The cost of the pipe laying in US$/m length can be determined from the linear formula 1200 × D where D is the pipe diameter expressed in metres; the friction factor of the pipe can be assumed at $\lambda = 0.02$.

The investment is going to be repaid from a 20-year loan with an interest rate of 8%. What will the annual repayments be if the total length of the pipe section is 1 km?

Answer:
The annuity calculated according to the conditions of the loan will be:

$$a_{20/8} = \frac{0.08 \times (1+0.08)^{20}}{(1+0.08)^{20} - 1} = 0.1019$$

From Equation 4.9, for $a = 1200$:

$$D = 0.05\sqrt{Q}\ \sqrt[6]{\frac{e}{\eta a_{n/r} a}}$$

$$= 0.05\sqrt{400}\ \sqrt[6]{\frac{0.15}{0.65 \times 0.1019 \times 1200}} = 0.352 \text{ m} \approx 350 \text{ mm}$$

If the pipe length is 1 km, the total investment cost can be estimated at 1200 × 0.35 × 1000 = 420,000 US$, which results in annual instalments of 0.1019 × 420,000 = 42,798 US$.

PROBLEM 4.3
For the same pipe diameter and length from Problem 4.2, calculate the annual loss of energy due to friction and its total cost.

Answer:
For pipe $D = 350$ mm, $L = 1000$ m and $\lambda = 0.02$, the friction loss from the Darcy–Weisbach Equation for flow $Q = 400$ m^3/h becomes:

$$\Delta H = \frac{\lambda L}{12.1 D^5} Q^2 = \frac{0.02 \times 1000}{12.1 \times 0.35^5}\left(\frac{400}{3600}\right)^2 = 3.89 \text{ mwc}$$

The energy wasted on the friction loss on an annual basis will be calculated as:

$$E = \frac{\rho g Q \Delta H}{1000 \times \eta} T$$

$$= \frac{1000 \times 9.81 \times 400 \times 3.89}{1000 \times 0.65 \times 3600} \times 24 \times 365 = 57{,}144 \text{ kWh}$$

and its annual cost will be EC $= 57{,}144 \times 0.15 = 8572$ US\$. This calculation has no practical meaning, as the loss of energy due to pipe friction is unavoidable. This loss can however be reduced by increasing the pipe diameter, which can help to reduce the pumping costs.

Self-study:
Spreadsheet lesson A5.1.7 (Appendix 5)

4.2 HYDRAULIC DESIGN

The hydraulic design of water transport and distribution systems requires thorough calculations due to the significant impact of each component on the overall operation. Opting for a larger diameter, reservoir volume or pump unit will always offer more safety in supply but implies a substantial increase in investment costs. This reserve capacity can only be justified by estimating the potential risks of irregular situations; otherwise the distribution system will become in part a dead asset causing considerable maintenance problems.

4.2.1 *Design criteria*

Hydraulic design primarily deals with pressures and hydraulic gradients. In addition, the flow velocities, pressure- and flow fluctuations are also relevant design factors.

The pressure criterion is usually formulated as the minimum/maximum pressure required, or allowed, at the most critical point of the system.

Table 4.2. Pressures in world cities (Source: Kujundžić, 1996).

City/Country	Min.–Max. (mwc)
Amsterdam/NL	± 25
Wien/Austria	40–120
Belgrade/Serbia	20–160
Brussels/Belgium	30–70
Chicago/USA	± 30
Madrid/Spain	30–70
Moscow/Russia	30–75
Philadelphia/USA	20–80
Rio de Janeiro/Brasil	±25
Rome/Italy	± 60
Sophia/Bulgaria	35–80

Minimum pressure requirements usually depend on company policy although they can also be standardised, i.e. prescribed by legislation. The starting point while setting the minimum pressure is the height of typical buildings present in the area, which in most urban areas consist of three to five floors. With pressure of 5–10 mwc remaining above the highest tap, this usually leads to a minimum pressure of 20–30 mwc above the street level. In the case of higher buildings, an internal boosting system is normally provided. In addition to this consideration, an important reason for keeping the pressure above a certain minimum can be fire fighting.

Maximum pressure limitations are required to reduce the additional cost of pipe strengthening. Moreover, there is a direct relation between (high) pressure and leakages in the system. Generally speaking, pressures greater than 60–70 mwc should not be accepted. However, higher values of up to 100–120 mwc can be tolerated in hilly terrains where pressure zoning is not feasible. Pressure reducing valves should be used in such cases. Table 4.2 shows pressure in the distribution systems of some world cities.

The table shows a rather wide range of pressures in some cases, which is probably caused by the topography of the terrain. In contrast, in flat areas such as Amsterdam, it is easier to maintain lower and stable pressures.

In distribution areas where drinking water is scarce, the pressure is not thought of as a design parameter. For systems with roof tanks, a few metres of water column is sufficient to fill them. However, in some distribution areas, even that is difficult to achieve and the pressure has to be created individually (as shown earlier in Figure 1.13).

Besides maintaining the optimum range, pressure fluctuations are also important. Frequent variations of pressure during day and night can create operational problems, resulting in increased leakage and malfunctioning of water appliances. Reducing the pressure fluctuations in the system is therefore desirable.

The design criteria for hydraulic gradients depend on the adopted minimum and maximum pressures, the distance over which the water needs to be transported, local topographic circumstances and the size of the network, including possible future extensions. The following values can be accepted as a rule of thumb:

- 5–10 m/km, for small diameter pipes,
- 2–5 m/km, for mid-range diameter pipes,
- 1–2 m/km, for large transportation pipes.

Velocity range can also be adopted as a design criterion. Low velocities are not preferred for hygienic reasons, while too high velocities cause exceptional head-losses. Standard design velocities are:

- $\pm$ 1 m/s, in distribution systems,
- $\pm$ 1.5 m/s, in transportation pipes,
- 1–2 m/s, in pumping stations.

4.2.2 *Basic design principles*

After the inventory of the present situation has been made, design goals have become clear and design parameters have been adopted, the next dilemma is in the choice of the supply scheme and possible layouts of the network. The following should be kept in mind while thinking about the first alternatives:

1. *Water flows to any discharge point choosing the easiest path: either the shortest one or the one with the lowest resistance.*
2. *Optimal design from the hydraulic perspective results in a system that demands the least energy input for water conveyance.*

Translated into practical guidelines, this means:

- maximum utilisation of the existing topography (gravity),
- use of pipe diameters that generate low friction losses,
- as little pumping as necessary to guarantee the design pressures,
- valve operation reduced to a minimum.

Yet, the hydraulic logic has its limitations. It should not be forgotten that the most effective way of reducing friction losses, by enlarging pipe diameters, consequently yields smaller velocities. Hence, it may appear difficult to optimise both pressures and velocities in the system. Furthermore, in systems where reliable and cheap energy is available, the cost calculations may show that the lower investment in pipes and reservoirs justifies the increased operational costs of pumping.

Hence, there are no rules of thumb regarding optimal pumping or ideal conveying capacity of the network. It is often true that more than one alternative can satisfy the main design parameters. Similar analysis as the one shown in Figure 4.2 should therefore be conducted for a number of viable alternatives, calculating the total investment and operational costs per alternative (instead of the pipe diameter, as the Figure shows). In any sensible alternative, larger investment costs will lead to lower operational costs; the optimal alternative will be the one where the sum of investment and operational costs is at a minimum.

The first step in the design phase is to adopt an appropriate distribution scheme. Pumping is an obvious choice in flat areas and in situations where the supply point has a lower elevation than the distribution area. In all other cases, the system may entirely, or at least partly, be supplied by gravity; these situations were discussed in Chapter 3.

The next step is in the choice of network configuration. Important considerations here are the spatial and temporal demand distribution and distances between the demand points, natural barriers, access for operation and maintenance, system reliability, possible future extensions, etc.

Water transport systems are commonly of a serial or branched type. Pipes will be laid in parallel if the consequences of possible failure affect large numbers of consumers, an industrial area or important public

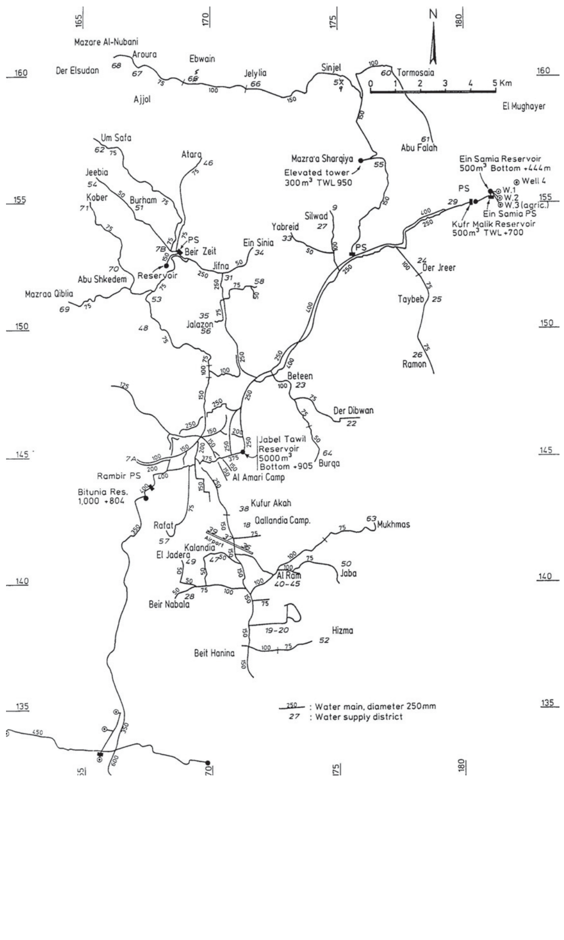


Figure 4.3. Branched water transport system in Palestine (Abu-Thaher, 1998).

complex (e.g. an airport). The layout of a water transport system often results from the existing topography and locations of the urban settlements. An example of a branched transportation system is shown in Figure 4.3 for the Ramallah-El Bireh district in Palestine. The system is located in a hilly area with elevations between 490 and 890 msl. It supplies approximately 200,000 consumers with an annual quantity of 9 million m^3 (Abu Thaher, 1998).

Creating loops is not typical for large transportation systems; such an approach is too expensive in many cases despite the shortcomings of the branched configuration. In smaller areas and with more favourable topographic conditions, this strategy may be feasible as it drastically improves the reliability of supply. An example from Figure 4.4 shows the regional system of the province of Flevoland in The Netherlands. The network of PVC pipes is laid in a sandy soil on a flat terrain; in 1996, it covered an area of approximately 230,000 consumers supplying an annual quantity of 15 million m^3. Some other water companies in The Netherlands also create loops in their transport systems; compared to the examples mentioned in Chapter 1, these are comparatively smaller transport systems and the 24-hour supply is a standard the Dutch consumers expect to be guaranteed for the price they pay for water.

Looped network configurations are common for urban distribution systems. How the layout should be developed depends on:
– the number and location of supply points,
– the demand distribution in the area,
– future development of the area.

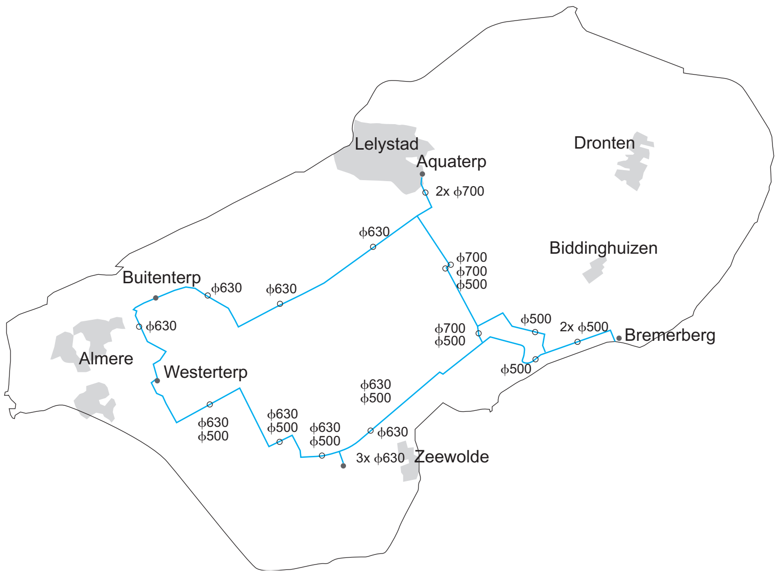


Figure 4.4. Looped water transport system (Province of Flevoland, 1996).

First, the backbone of the system, made of large pipe diameters (secondary mains), has to be designed. If the network is supplied from one side, this can be of a branched structure. Characteristic of such a system is that the pipe diameters will gradually reduce towards its end. A problem occurs if an alternative source, considered for future supply, is located on the opposite side of the network. Forming a loop (a so-called ring) or a few major loops of the secondary mains is a better solution for this sort of problem, although more expensive.

The secondary mains often follow the routes of the main streets in the area, for the sake of easier access for maintenance and repair. A good starting point while selecting the main structure of the network is to examine the paths of the bulk flows, which can be determined for a known demand distribution in the area. If a network computer model is available, a preliminary test can be conducted by assuming uniform diameters in the system. The result of such simulation would show larger friction losses (velocities) in pipes carrying more water, indicating them as potential secondary mains.

An example of the distribution network with a skeleton of the secondary mains is presented in Figure 4.5, for Zadar, a town in the coastal zone in Croatia. The gravity system supplies between 75,000 and 125,000 consumers (during the tourist season) with an average annual quantity of 8 million m^3 (Gabrić, 1997).

In the second stage, the sizing of distribution pipes and analysis of the network hydraulic behaviour takes place. The support of a network computer model is fundamental here: the weak points in the system are easy to detect, and it is possible to anticipate the right type and size of the pumps and reservoirs needed in the system, as well as the additional pipe connections required. Alternatives that satisfy the main design criteria can further be tested on other aspects, such as operation under

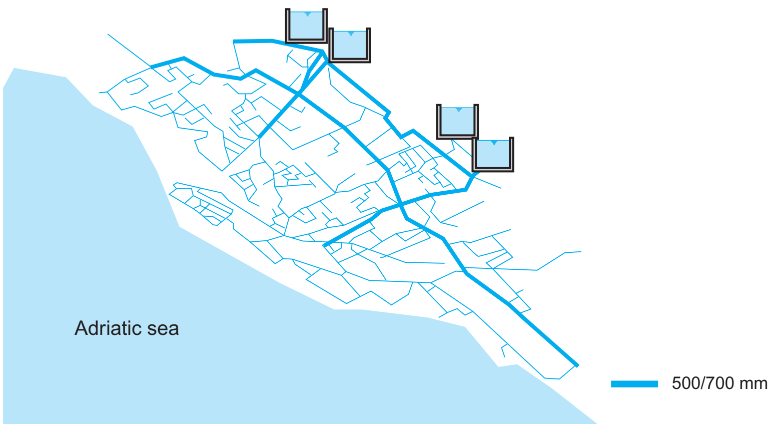


Figure 4.5. Layout of distribution network of Zadar, Croatia (Gabrić, 1997).

irregular situations, system maintenance, possible water quality deterioration, etc.

Transportation pipes that supply balancing reservoirs in the system are commonly designed for average flow conditions on the maximum consumption day. In distribution systems where 24-hour supply is a target, the network will be sized for the maximum consumption hour of the maximum consumption day. The ultimate buffer for safety is provided if, on top of that, a calamity situation is assumed to take place at the same moment: a fire or a failure of any of the system components. As mentioned in Chapter 2 however, it may be more cost effective to let a limited number of consumers 'enjoy' somewhat lower pressure or even an interruption over a short period of time, rather than to specify pipes of a few per cent larger diameter in considerable parts of the network in order to prevent a relatively rare problem occurring. Such considerations constitute part of the reliability analysis of the system, which is elaborated further in Chapter 6.

Finally, the fire demand requirement is usually a dominant factor that influences the size of the pipes; in smaller pipe diameters it is actually a major contributor to the peak demand compared to the regular demand. To avoid oversized systems, the pipe diameters can be adopted based on the average hour instead of the maximum hour demand on the maximum consumption day, in addition to the fire demand. For instance, this is a common practice of many water companies in USA, which seems to offer a good balance between the investment and the reliability concerns.

The points made in this and the remaining sections of Paragraph 4.2, are illustrated in a simplified design case of a medium size town, discussed in detail in Appendix 2. The electronic materials on the attached CD can be used for a better understanding of the exercise; the instructions for their use are also given in Appendix 6. A cost comparison of the two developed design alternatives has been conducted according to the present worth method, discussed in Section 4.1.2.

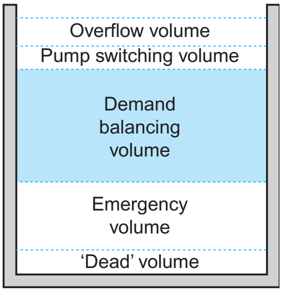


Figure 4.6. Volume requirements in a reservoir.

4.2.3 *Storage design*

While designing a storage volume, provision should be planned as in Figure 4.6.

Demand balancing volume

The *demand balancing volume* depends on demand variations. A 24-hour demand balancing is usually considered by assuming constant (average) production feeding the tank, and variable demand supplied from it (Figure 3.2). Unless assumed to be constant, leakage should also be included in this balancing.

The calculation is based on the tank inflow/outflow balance for each hour. Cumulative change in the tank volume (Equation 3.4) can be

Table 4.3. Example of the determination of the balancing volume.

Hour	Q(m^3/h)	*pf*	1−*pf*	Σ(1−*pf*)
1	579	0.39	0.61	0.61
2	523	0.35	0.65	1.26
3	644	0.43	0.57	1.83
4	835	0.56	0.44	**2.27**
5	1650	1.11	−0.11	2.16
6	1812	1.22	−0.22	1.94
7	1960	1.31	−0.31	1.63
8	1992	1.34	−0.34	1.29
9	1936	1.30	−0.30	0.99
10	1887	1.27	−0.27	0.72
11	1821	1.22	−0.22	0.50
12	1811	1.22	−0.22	0.28
13	1837	1.23	−0.23	0.05
14	1884	1.27	−0.27	−0.22
15	2011	1.35	−0.35	−0.57
16	2144	1.44	−0.44	−1.01
17	2187	1.47	−0.47	−1.48
18	2132	1.43	−0.43	−1.91
19	1932	1.30	−0.30	**−2.21**
20	1218	0.82	0.18	−2.02
21	898	0.61	0.39	−1.63
22	786	0.53	0.47	−1.16
23	657	0.44	0.56	−0.60
24	601	0.40	0.60	0
Avg	1489	1		

observed, and the total balancing volume required is going to comprise the two extremes:

– the maximum accumulated volume stored when demand drops below average,
– the maximum accumulated volume available when demand is above average.

The procedure is illustrated in Table 4.3 for the diurnal demand pattern shown in Figure 4.7. The equal areas 1 and 2 in the figure are proportional to the balancing volume of the tank.

From the table: $V_{bal} = (2.27 + 2.21) \times 1489 = 6671\ m^3$, which is 18.6% of the total daily demand of 35,737 m^3. The balancing volume (and therefore the total volume) is at its maximum at the end of hour 4 (4.00 a.m.). During the next hour, the diurnal peak factor becomes greater than 1 and the tank will start to loose its volume until the moment the peak factor drops below 1 again. This happens at the end of hour 19 (7.00 p.m.), when the balancing volume is completely exhausted. During the rest of the period the volume of the tank will be replenished back to the initial level at the beginning of the day. The required balancing

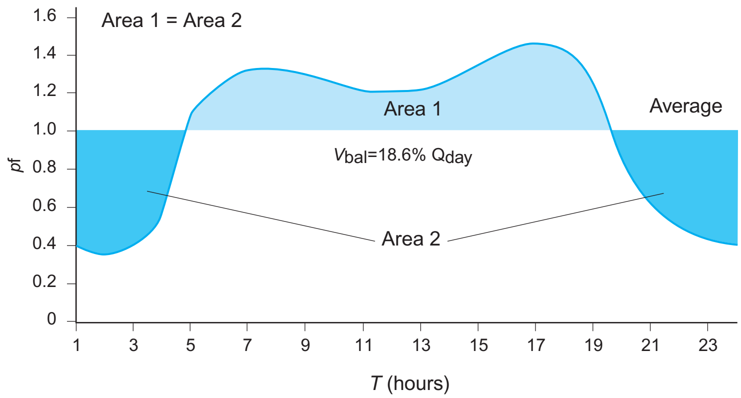


Figure 4.7. Relation between the demand pattern and balancing volume.

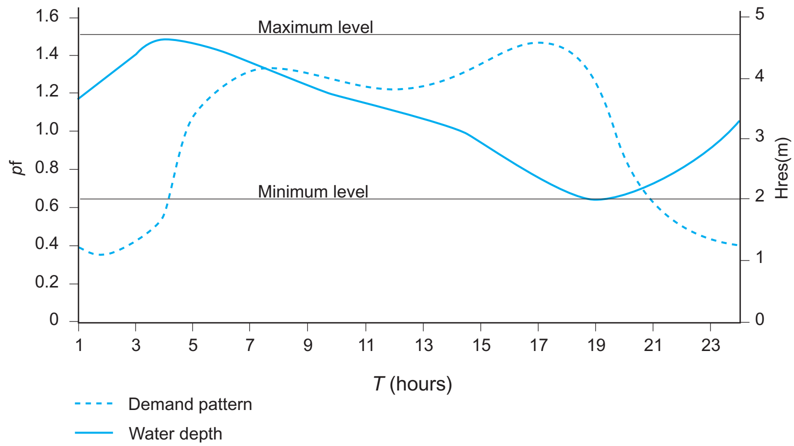


Figure 4.8. Relation between the demand pattern and reservoir water level variation.

volume at that moment is: $V_0 = 2.21 \times 1489 = 3291\ \text{m}^3$. Assuming a cross-section area $A = 2500\ \text{m}^2$ and the minimum depth (incl. the reserve volume), $H_{min} = 2$ m, the level variation in the tank will be as shown in Figure 4.8.

Depending on the shape of the demand pattern, the balancing volume usually takes between 10% and 30% of the maximum day consumption. Generally smaller volumes are needed:

– for flat diurnal patterns,
– for diurnal patterns which fluctuate around the average flow,
– if pumps in the system are operated to follow the demand pattern to some extent.

Examples of these cases are shown in Figures 4.9–4.11, respectively.

In the last diagram, the demand variation is balanced predominantly by operating the pumps. Four equal units connected in parallel, each of them supplying 40% of the average flow, are used to deliver the hourly

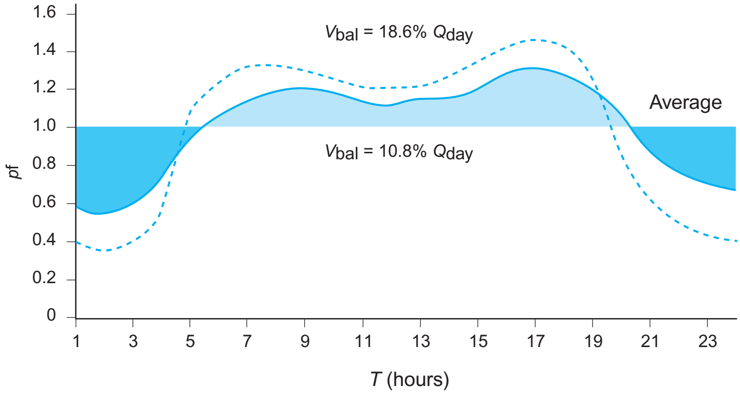


Figure 4.9. Balancing volume in the case of a flat diurnal pattern.

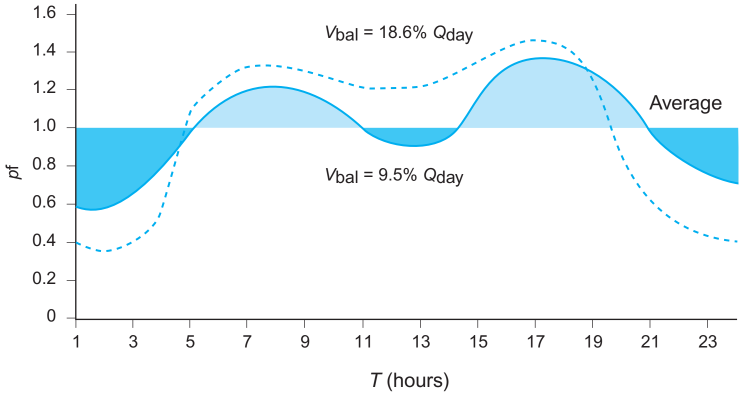


Figure 4.10. Balancing volume in the case of a fluctuating diurnal pattern.

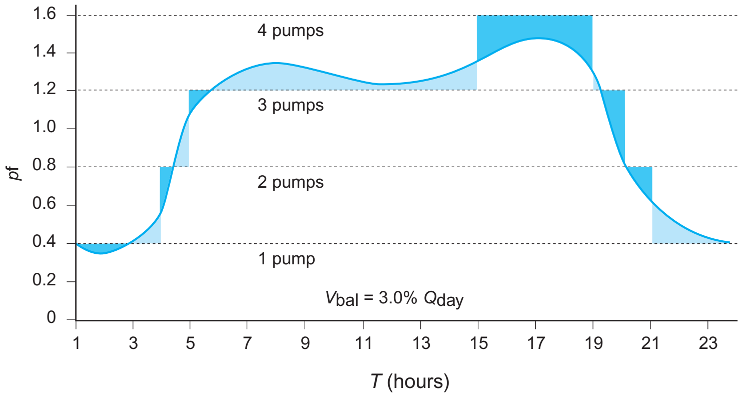


Figure 4.11. Balancing volume in the case of scheduled pumping.

demand. The first pump is in operation between hours 1 and 4 (1.00 a.m.–4.00 a.m.), when the second unit is switched on. An hour later, the third unit starts operation and from hour 15 (3.00 p.m.) all 4 pumps are 'on'. This mode will continue until subsequent switching of

3 pumps takes place at hours 19, 20 and 21 (7.00 p.m.–9.00 p.m.). The tank volume in this set-up is used for optimisation of the pumping schedule rather than to balance the entire demand variation. Without the tank, the fourth unit would have to operate for much longer, at least from 6.00 a.m., in order to guarantee the minimum pressures in the system; other units would have to change their operation, too.

This example is typical for the operation of water towers. From the perspective of energy consumption, this is usually a more expensive solution than to pump the average flow continuously over 24 hours but the investments costs of the reservoir volume will be minimised. Hence, *the smaller the balancing volume is, the more pumping energy will be required*.

Applying a similar concept to that in Table 4.3, the required balancing volume can also be determined graphically. If the hourly water demand is expressed as a percentage of the total daily demand, it can be plotted as a cumulative water demand curve that will be compared to a corresponding cumulative supply curve.

In the example in Figure 4.12, for a constant-rate supply over 24 hours, a straight line will represent the supply pattern. The required balancing volume equals the sum of the two extreme distances between the demand and supply curves (A–A′ plus B–B′), which is about 28% of the daily demand. The balancing volume available at the beginning of the day should equal the B–B′ percentage. 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.

If the supply capacity is so high that the daily demand can be met with 12 hours of pumping a day, the required storage is found to be C–C′

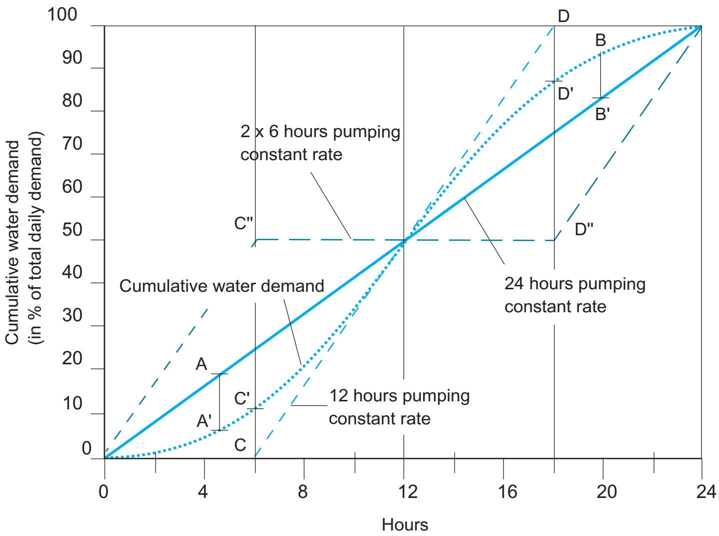


Figure 4.12. Example of graphical determination of the balancing volume (IRC, 2002).

plus D–D′, which in this case is 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 a cheaper tariff), the required balancing volume will have to be much bigger. In the case of pumping between 6.00 p.m. and 6.00 a.m., the balancing volume becomes C′–C″+ D′–D″≈76% of the daily demand. Hence, the *time period of intermittent pumping has implications for the size of the balancing volume*.

The volumes calculated as explained earlier (except for the water tower) are the volumes that balance the demand of the entire distribution area. These volumes can be shared between a few reservoirs, depending on their elevation and pumping regimes in the system. Optimal positioning and size of these reservoirs can effectively be determined with the support of a computer model. As Figure 4.13 shows, a correctly located reservoir more or less repeats the same water level pattern every 24 hours. A reservoir located too low soon becomes filled with water due to excessive pumping, while a reservoir located too high is going to dry out after some time due to insufficient pumping.

If better positioning of the tank is impossible, the pumping regime should be adjusted to correct the reservoir balancing. Such a measure will obviously have implications for pressure in the system.

Besides the balancing volume, other provisions in a reservoir include emergency volume, 'dead' volume, overflow volume and pump switching volume.

Emergency volume

Emergency volume is exclusively used outside the regular supply conditions:
– during planned maintenance of the system,
– during a failure either in production facilities or somewhere else in the network,
– for fire fighting requirements.

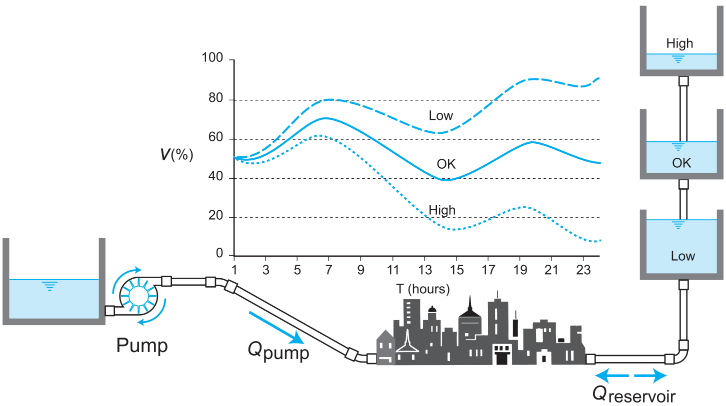


Figure 4.13. Relation between a tank's water level pattern and its altitude.

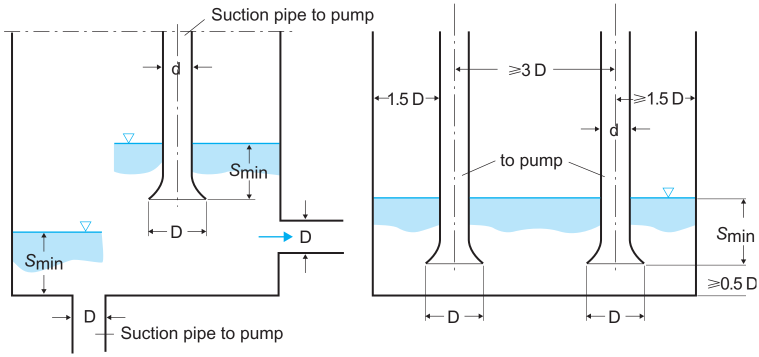


Figure 4.14. Minimum reservoir level where pumping is involved (KSB, 1992).

How much water should be reserved depends primarily on how quickly the cause of interruption can be put under control. Each hour of average flow supply requires a volume equal to 4–5% of the (maximum) daily demand. A few hours' reserve is reasonable, more than that increases the costs of the tank, which also creates problems from water stagnation. Despite that, huge emergency volumes can be planned in large distribution areas. Special precautions then have to be taken in maintaining the water quality in the tank (discussed further in Section 4.5.9).

'Dead' volume

'Dead' volume is never used. It is provided as a reserve that should prevent the reservoir from staying dry. ±15 cm of the depth is usually reserved for the 'dead' volume. More than that might be necessary if pumps that are supplied by the tank are located above the minimum water level. Certain provisions are required in that situation in order to prevent under-pressure in the suction pipe. The guideline suggested by the KSB pump manufacturer is presented in Figure 4.14. S_{min} from the figure equals $v^2/2g+0.1$ m, where v is the maximum velocity in the suction pipe.

Overflow volume

An *overflow volume* is provided as a protection against reservoir overflow. 15–20 cm of the depth can be allocated for that purpose. Within that range, the float valve should gradually close the inlet. For added safety, an outlet arrangement that brings the surplus water out of the system should be installed.

Pump switching volume

Pump switching volume is necessary if corresponding pumps operate automatically on level variation in the tank. There is a potential danger if the pump switches-on and -off at the same depth: switching may happen too frequently (e.g. more than once every 15 minutes) if the water level fluctuates around this critical depth. To prevent this, the switch-on and -off depths should be separated (see Figure 4.15). Depending on the volume of the tank, 15–20 cm of the total depth can be reserved for this purpose.

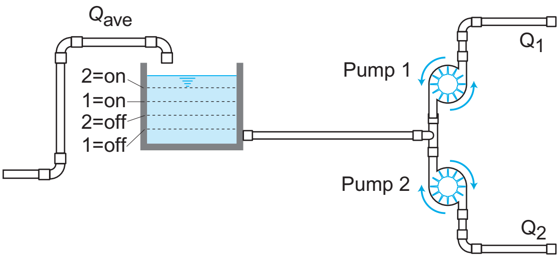


Figure 4.15. Pump switching levels in the reservoir.

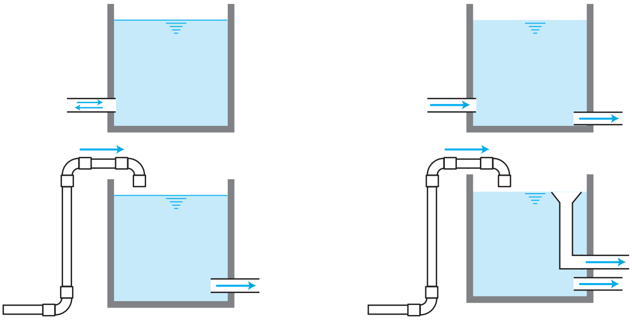


Figure 4.16. Reservoir inlet and outlet arrangements.

The hydraulics in the system may have an impact when selecting the inlet and outlet arrangements of a reservoir. Some examples are shown in Figure 4.16. The inflow from the top prevents backflow from the reservoir, while the outflow from the top usually serves as the second outlet, against overflow.

Self-study:
Workshop problems A1.5.8–A1.5.10 (Appendix 1)
Spreadsheet lesson A5.8.10 (Appendix 5)

4.2.4 *Pumping station design*

The capacity of a pumping station is usually divided between several units that are connected in parallel. A typical set-up consists of the elements shown in Figure 4.17.

The role of particular components is as follows:

1 Valves are commonly installed at both the suction- and pressure-side of the pump. These are used if the pump has to be dismantled and removed for overhaul or replacement. If necessary, a bypass can be used while this is being carried out. During regular operation of fixed speed pumps, the valve on the pressure side is sometimes throttled if the pumping head is too high.
2 A non-return valve on the pressure side serves to prevent reverse flow.
3 An air valve on the pressure side is used to purge air out of the system.

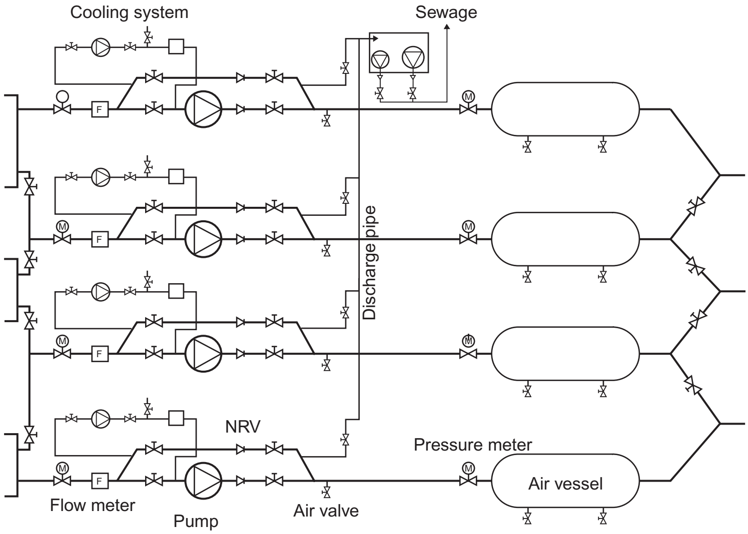


Figure 4.17. Pumping station layout.

Water hammer

4 The air vessel on the pressure side dampens the effects of transient flows that appear as soon as the pump is switched on or off, causing a pressure surge known as *water hammer*.
5 Measuring equipment: to register the pumping head, pressure gauges will be installed on both sides of the pump. A single flow meter is sufficient.
6 A cooling system is installed for cooling of the pump motors.
7 Discharge pipes allow the emptying of the entire installation if needed for maintenance of the pipelines.

The following main goals have to be achieved by the proper selection of pump units:
– high efficiency,
– stable operation.

Furthermore, the selected pumps should preferably have the similar number of working hours. This is easy to achieve if all pumps are of the same model, which allows their schedules to be rotated.

In theory, the pumps that deliver duty head and duty flow are assumed to operate with optimal efficiency. These two parameters are used for the preliminary selection of pump units. The initial choice can be made from a diagram as shown in Figure 4.18. Such diagrams, showing operating ranges of various models, are commonly available by pump manufacturers.

It is possible to determine the impeller diameter, available net positive suction head and required pump power from the graphs related to a particular type (see Figure 4.19).

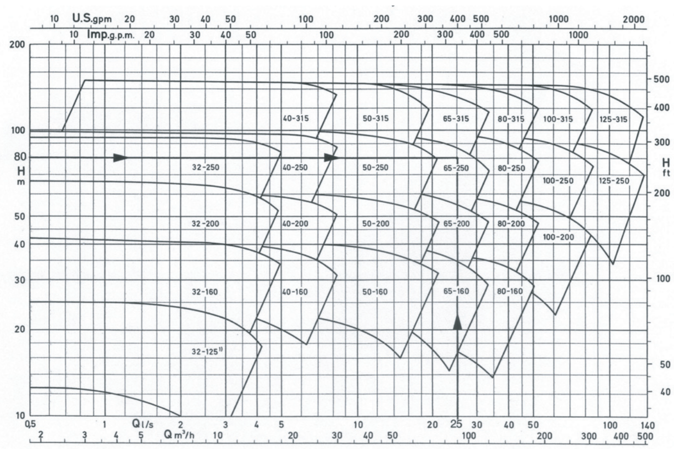


Figure 4.18. Operational regimes of pumps (KSB, 1992).

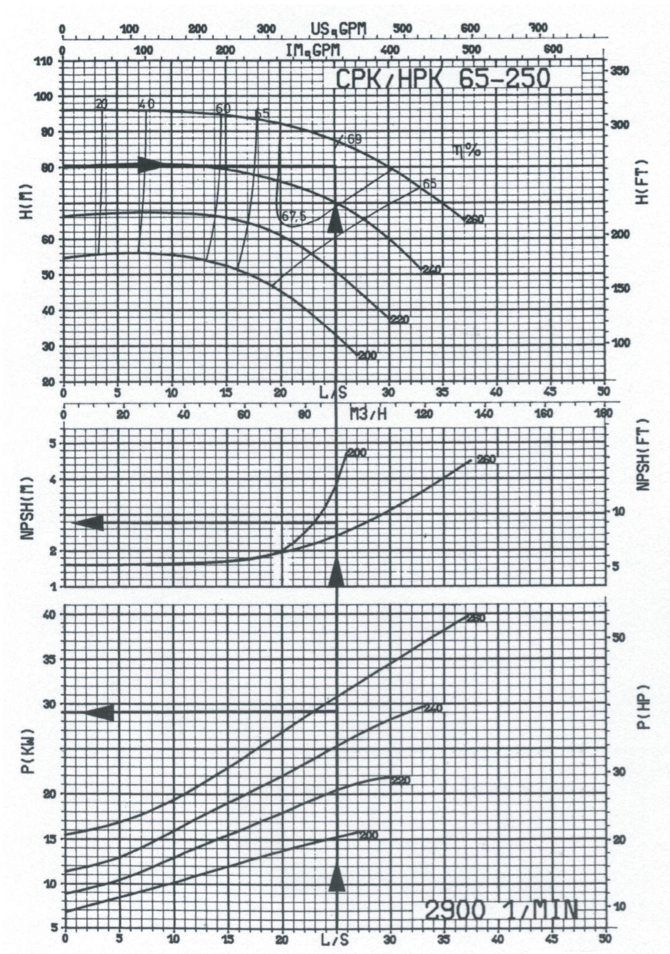


Figure 4.19. Selection of pump type (KSB, 1992).

The pump power can also be calculated using Equation 3.48. In this case the values for the design head and flow assume the pump is operating under the maximum expected flow. A 10–15% safety margin is normally added to the result; the first higher manufactured size will be adopted.

Rated power

For pumps driven by electrical motor, a transformer has to be sized. The capacity in kVA is calculated from the *rated power*:

$$N_{\text{rated}} = \frac{\sum_{i=1}^{n} N_{m,i} + N_{\text{eq}}}{\cos\ \theta} \tag{4.10}$$

where n is the number of pumps in operation under maximum supply conditions, $N_{m,\ i}$ is motor power per unit, calculated by Equation 3.49 in kW, N_{eq} is provision for other equipment in the pumping station: light, welding corner, etc. and cos θ is the *power factor*, which takes a value of between 0.7–0.8.

Power factor

If a diesel generator is to be provided in the pumping station, its size will be designed to cover an electricity failure assumed to take place during the maximum supply conditions. With efficiency η_d, the generator power can be calculated from the following formula:

$$N_d = \frac{\sum_{i=1}^{n} N_{p,i} + N_{\text{eq}}}{\eta_d} \tag{4.11}$$

More energy is needed to start the pumps by diesel engine than by electricity. To be on the safe side, the pump power of the largest unit, $N_{p,\ 1}$, in Equation 4.11 is assumed to be doubled. Finally, the power needed to start the engine is:

$$N_{\text{rated}} = \frac{N_d}{\cos\ \theta} \tag{4.12}$$

Cavitation

The net positive suction head (NPSH) is the parameter used for risk analysis of cavitation. This phenomenon occurs in situations when the pressure at the suction side of the pump drops below the *vapour pressure*.[1] As a result, fine air bubbles are formed indicating the water is boiling at room temperature. When the water moves towards the area of high

[1] Water starts to boil when its vapour pressure reaches the surrounding pressure that is dependant on altitude and affects the boiling temperature. As is well known, at mean sea level and normal atmospheric pressure, water boils at 100 ºC. At higher altitudes the atmospheric pressure becomes lower and water will start to boil at a lower temperature.

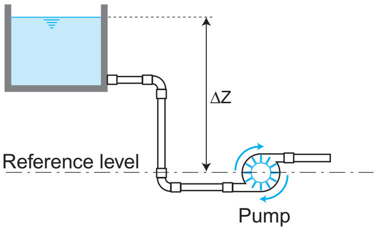


Figure 4.20. Pump unit located below the suction level in the reservoir.

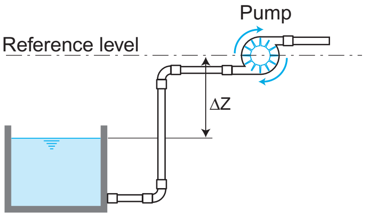


Figure 4.21. Pump unit located above the suction level in the reservoir.

pressure, i.e. to the area around the impeller, the bubbles suddenly collapse causing dynamic forces, ultimately resulting in pump erosion. The damage becomes visible after the pump has been in operation for some time, and causes a reduction of the pump capacity (the actual pumping curve shifts lower than the original one).

The available NPSH is determined based on the elevation difference between the pump impeller axis and the minimum water level at the suction side. Two possible layouts are shown in Figures 4.20 and 4.21.

The following simplified equation is used in practice, which is valid for common range of altitudes and water temperatures:

$$NPSH_{available} = 10 - \Delta H \pm \Delta Z \tag{4.13}$$

where ΔH is the total head-loss along the pipe section between the reservoir and the pump (mwc) and ΔZ is elevation difference between the pump axis and minimum suction level (m); the value of ΔZ becomes negative if the pump axis is located above the suction level.

To prevent cavitation, the available NPSH has to be greater than the minimum NPSH required for the pump, which is read from diagrams such as the one in Figure 4.19. A safety margin of 0.5–1.5 m is normally added to the calculation result.

To keep the head-losses reasonably low, pipes in pumping stations are designed to maintain the optimal range of velocities; the recommended values are:

– feeder main $\upsilon = 0.6–0.8$ m/s
– suction pipe $\upsilon = 0.8–1.2$ m/s

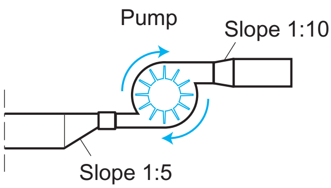


Figure 4.22. Pipe reducers.

– pressure pipe v = 1.5–2.0 m/s
– discharge header v = 1.2–1.7 m/s

The total head-loss of a few metres of water column is common in pumping stations. In addition to pipe friction, this is the result of lots of valves being installed, and bends created in order to 'pack' the pipes within a relatively small space. To be able to limit the energy losses and meet the NPSH requirement, the reducers and enlargers will be constructed to reduce the pipe velocity; the recommended slopes are shown in Figure 4.22.

The critical head-loss is calculated for the worst positioned pump unit, usually the last one in the pump arrangement, and under the maximum supply conditions. The friction and minor losses will be calculated according to the standard procedures explained in Paragraph 3.2. Detailed tables for calculation of the minor losses are available in Appendix 3. Unless stated otherwise, all minor loss factors there, are given for downstream flow velocity (after the obstruction).

4.3 COMPUTER MODELS AS DESIGN TOOLS

Some 20 years ago, computer modelling of water distribution networks was carried out to only a very limited extent. The massive introduction of personal computers in the early nineties changed this situation entirely. The commercial programmes available on the market nowadays enable very accurate and quick calculations, even for networks consisting of thousands of pipes. These programmes are effective for use in the network design as well as for analyses of the system operation and planning of its maintenance.

The model of the Amsterdam network shown in Figure 4.23 was prepared using the 'InfoWorks WS' software, developed by Wallingford Software Ltd of the UK. The programme takes just a few minutes to complete a 24-hour simulation of the distribution model consisting of nearly 40,000 pipes, which is a standard calculation speed for the latest generation of software. Within such a short period of time, the entire system is calculated not once but 96 times, for every 15 minutes of the

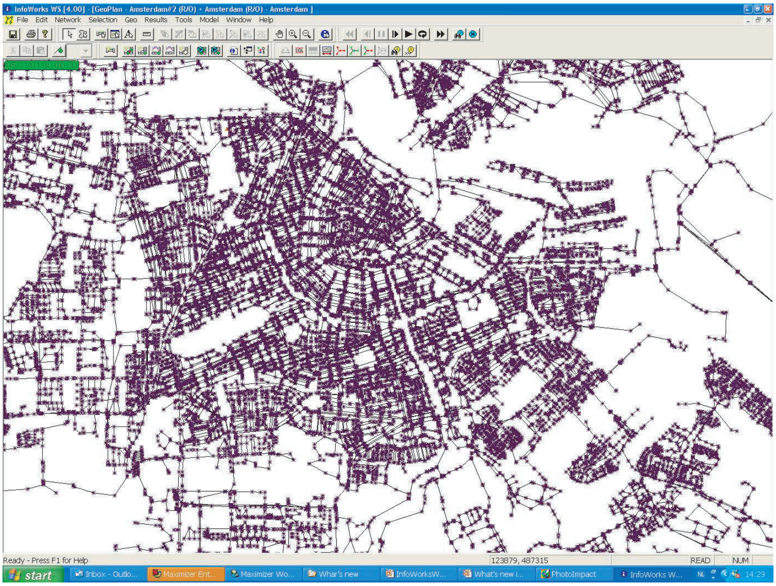


Figure 4.23. Computer model of the Amsterdam distribution network (Source: Wallingford Software Ltd).

operation during a particular day! It is obviously ridiculous to compare the time used for manual calculation against the time used for computer modelling, as they are on completely different scales.

Modern commercial distribution network programmes are rather similar in concept, having the following common features:

– they have the demand-driven calculation modules generally based on the principles explained in Paragraph 3.5,
– they allow extended period hydraulic simulations,
– they possess an integrated module for water quality simulations,
– they can handle a network of virtually unlimited size,
– they can calculate complex configurations in a matter of minutes,
– they have excellent graphical interface for the presentation of results.

The main distinctions are in the specific format of input data used, as well as in the way the calculation results are processed.

Most of the algorithms on which the programme engine is based simulate the network operation by calculating a number of consecutive steady states. This number will be defined by selecting a uniform time interval between them, usually between a few minutes and up to one hour. Adjustment of the input data before each new calculation is done automatically, based on the results of the previous steady state calculation. Having the type and operation of the network components specified for the entire simulation period makes the programme able to read the peak demands, recalculate storage volumes, switch the pumps-on or -off, etc. The final outputs of the simulation are the diagrams that describe the pressure-, surface level- and flow-variations in the system. The hydraulic results are further used as an input for the water quality simulation.

The entire modelling process consists of the following steps:

1 input data collection,
2 network schematisation,
3 model building,
4 model testing,
5 problem analysis.

4.3.1 *Input data collection*

By possessing powerful computational tools, the focus in modelling has shifted from the calculation to the collection of reliable input data. High quality information concerning demand, system dimensions, materials and the maintenance level is crucial for accurate results; *quality of the input = quality of the output*. 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*
  – Layout of the system – pipe routes and junctions; location of the main components.
  – Topography – ground elevations in the area of the system; some specific natural barriers.
  – Type of the system – distribution scheme: gravity, pumping, combined; role of each system component; pressure zones.
  – Population – distribution and estimated growth.

2 *Water demand*
  – Demand categories present in the system: domestic, industry, tourism, etc.
  – Average consumption, patterns of variation: daily, weekly, and seasonal.
  – Type of domestic water use: direct supply, roof tanks, etc.; average household size, habits with respect to water use.
  – Demand forecasting.

3 *Network layout*
  – 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) and
    • average consumption and dominant categories.
  – Pipes – concerns predominantly the pipes $D \geq 80$–100 mm. Relevant for each pipe are:
    • length,
    • diameter (internal),

  - material and age,
  - assessment of corrosion level (k, C or N value if available).
- Service reservoirs – type (ground, elevated), capacity, minimum and maximum water level, shape (e.g. described through the volume–depth curve), inlet/outlet arrangement.
- Individual roof tanks (where applicable) – type and height of the tank, capacity, inflow/outflow arrangements, average number of users per house connection, description of house installations (existence of direct supply in the ground floor).
- Pumping stations – number and type (variable, fixed speed) of pumps; duty head and flow and preferably the pump characteristics for each unit; age and condition of pumps; efficiency and energy consumption.
- Others – description of appurtenances that may significantly influence the system operation (e.g. valves, measuring equipment, etc.)

4 *System operation and monitoring*
Important (and preferably simultaneous) measurements for calibration of the model are:
- the pressure in a number of points covering the entire network,
- level variations in the service reservoirs and roof tanks (where applicable),
- pressures and flows in the pumping stations,
- the flows in a few main pipes in the network,
- valve operation (where applicable).

In modern water distribution companies, much of the required information is directly accessible from the on-line monitoring of the system. In many networks in the developing world, a large part of this information is missing or incomplete and the only real source available is the operator in the field. Even without lots of measuring equipment, some knowledge of the system is likely to exist in a 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 sufficient pressure, etc. Where there is a possibility of continuous measurements, typical days should be compared: the same day of the weok in various seasons, or various days of the week in the same season.

5 *System maintenance*
The main aspects to be analysed include common maintenance practices, water metering, the UFW level and structure and water composition in the distribution network.

6 *Water company*
Organisation, facilities, practices (such as equipment and personnel for rapid reaction in calamity situations), plans for future extension of the system, etc.

4.3.2 *Network schematisation*

Network skeletonisation

The hydraulic calculation of looped networks is based on a system 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. This was particularly important in former times, when computers were not extensively used and schematising of the network could save several days (or even weeks) of calculation. In the meantime, this has become a minor problem and current development of the network modelling takes the opposite direction; with the introduction of modern data bases such as geographical information systems (GIS), more and more detailed information is included in analyses, specifically for monitoring of the network operation. Nevertheless, the user is now confronted with a huge amount of data resulting in somewhat bulky computer models, not always easy to handle or understand properly. Schematisation therefore still remains an attractive option in situations where the design of the main system components is analysed, because:
– it saves computer time,
– it allows model building in steps i.e. easier tracing of possible errors,
– it provides a clearer picture about global operation of the system.

Complex models of more than a few 100 pipes are not relevant for the design of distribution systems. Modelling of pipes under 100 mm significantly increases the model size without real benefit for the results. It also requires much more detailed information about the input, which is usually lacking. Thus, it is possible for the larger model to be less accurate than the small one.

The most common means of the network schematisation are:
– combination of a few demand points close to each other into one node,
– exclusion of a hydraulically irrelevant part of the network such as branches and dead ends at the borders of the system,
– neglecting small pipe diameters,
– introduction of equivalent pipe diameters.

On the other hand, while applying the schematisation it is not allowed to:
– omit demand of excluded parts of the network,
– neglect the impact of existing pumps, storage and valves.

The decision on how to treat the network during the process of schematisation is based on the hydraulic relevance of each of its components. It is sometimes desirable to include all main pipes; in other situation pipes under a specific size can be excluded (e.g. below 200 mm). As a general guideline, small diameter pipes can be omitted:
– when laying perpendicular to the usual direction of flow,
– if conveying flows with extremely low velocities,

Figure 4.24. Example of network schematisation.

– when located in the vicinity of large diameter pipes,
– when located far away from the supply points.

Whatever simplification technique is applied, the basic structure of the system should always remain intact, without removing the pipes that form the major loops. If the process has been properly conducted, the observed results of the schematised and full-size model for different supply conditions should deviate by not more than a few per cent.

An example of the network schematisation shows the simplified network in Hodeidah (already displayed in Figure 1.9) used for a computer model in a hydraulic study. Figure 4.24 shows the same network when pipes of 100, 200 and 300 mm diameters are removed successively, and finally the schematised layout that was used in hydraulic calculations.

4.3.3 *Model building*

Computer programmes for network hydraulic modelling distinguish between two general groups of input data:
1 Junctions – describing sources, nodes and reservoirs (water towers),
2 Links – describing pipes, pumps and valves.

Although the way some components are modelled may differ from one to another software, the following input information is required in all cases:
- *Sources*: identification, location and elevation of water surface level.
- *Nodes*: identification, location and elevation, base demand and pattern of demand variation.
- *Reservoirs*: identification, position, top and bottom water level, description of the shape (cross-section area, either the volume–depth diagram), initial water level at the beginning of the simulation, inlet/outlet arrangement.
- *Pipes*: identification, length, diameter, description of roughness, minor loss factor.
- *Pumps*: identification, description of pump characteristics, speed, operation mode.
- *Valves*: identification, type of valve, diameter, head-loss when fully open, operation mode.

For simulations of water quality, additional input information is required, such as: initial concentrations, patterns of variation at the source, decay coefficients, etc. Finally, a number of parameters which control the simulation run itself, have to be specified in the input: duration of the simulation, time intervals, accuracy, preferred format of the output, etc.

Based on the earlier input, the raw results of hydraulic simulation are flow patterns for links, and piezometric heads recalculated into pressures and water levels for junctions.

In addition, the water quality simulations offer the following patterns in each junction:
- concentration of specified constituent,
- water age,
- mixing of water from different sources.

In most cases, the input file format has to be strictly obeyed; this is the only code the programme can understand while reading the data. There are scarcely ever two programmes with exactly the same input format, so the chance of making errors during the model building is very likely. Recent programmes allow input in an interactive way, which reduces the chance for error caused by false definition of the network (see Figure 4.25). The disadvantage here is that the input data become scattered behind numerous menus and dialog boxes, often for each individual element of the network, which makes omission of some information fairly probable. Nevertheless, the testing of the network prior to the calculation is a standard part of any commercial software nowadays, and necessary feedback will be sent to the user depending on the library of error and warning messages.

Just as with networks in reality, *it is advisable to build the model in steps, gradually increasing the level of detail*. Starting immediately with

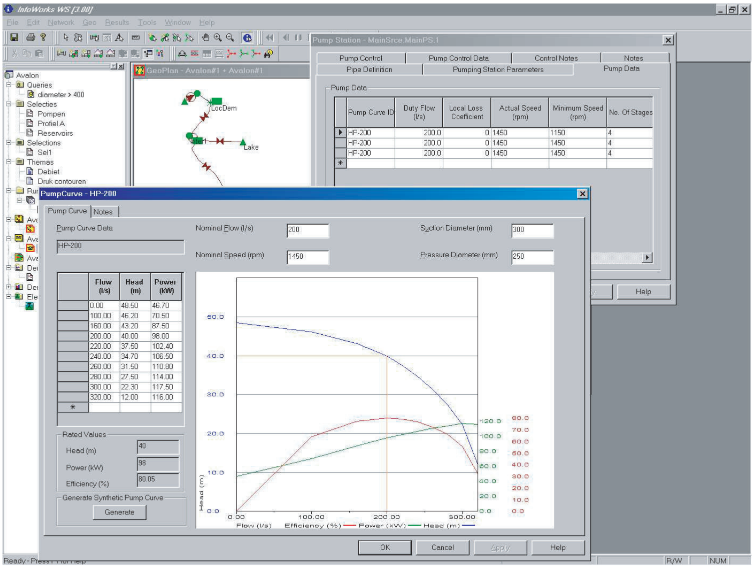


Figure 4.25. InfoWorks WS – interactive data input (Source: Wallingford Software Ltd).

the full size network, with all components included, will most likely yield problems during the model testing procedure.

4.3.4 *Nodal demands*

A special aspect of the model building process is the determination of nodal demands. The problem arises from the need to survey numerous users spread all over the network and concentrate their demand into a limited number of pipe junctions in order to make the network presentation suitable for a computer model.

The starting point is the average demand calculation carried out with Formulas 2.7–2.11. The formulas yield the demand of a certain area, which has to be converted into demand at a point (pipe junctions). The next step is the conversion procedure based on the following assumptions:

– an even distribution of consumers,
– the border between the supply areas of two nodes connected by a pipe is at the half of their distance.

A unit consumption per metre of the pipe length can be established for each loop formed by m pipes:

$$q_l = \frac{Q_l}{\sum_{j=1}^{m} L_{j,l}} \tag{4.14}$$

Q_l is the average demand within loop l, and L_j the length of pipe j forming the loop. Each pipe supplies consumers within the loop by a flow equal to:

$$Q_{j,l} = q_l \times L_{j,l} \tag{4.15}$$

and node i, connecting two pipes of loop l, will have the average consumption:

$$Q_{i,l} = \frac{Q_{j,l} + Q_{j+1,l}}{2} \tag{4.16}$$

One pipe often belongs to two neighbouring loops i.e. one node may supply the consumers from several loops. The final nodal consumption is determined after the above calculation has been completed for all loops in the system:

$$Q_i = \sum_{l=1}^{n} Q_{i,l} \tag{4.17}$$

n denotes the number of loops supplied by node i.

The procedure is illustrated in the example of two loops, shown in Figure 4.26.

Average demands in areas A and B and lengths of the pipes are known data. Furthermore:

$$q_A = Q_A/(L_{1-2} + L_{4-5} + L_{1-4} + L_{2-5})$$
$$q_B = Q_B/(L_{2-3} + L_{5-6} + L_{2-5} + L_{3-6})$$

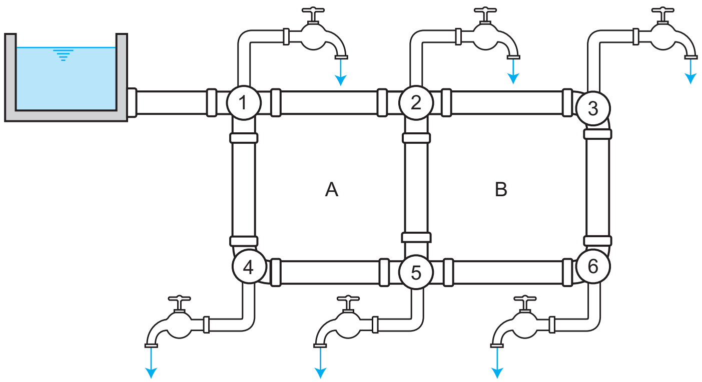


Figure 4.26. Example of nodal demands.

In loop A, the pipes supply:

$$Q_{1-2} = q_A \times L_{1-2}$$
$$Q_{4-5} = q_A \times L_{4-5}$$
$$Q_{1-4} = q_A \times L_{1-4}$$
$$Q_{2-5,A} = q_A \times L_{2-5}$$

and in loop B:

$$Q_{2-3} = q_B \times L_{2-3}$$
$$Q_{5-6} = q_B \times L_{5-6}$$
$$Q_{2-5,B} = q_B \times L_{2-5}$$
$$Q_{3-6,A} = q_B \times L_{3-6}$$

Pipe 2–5 appears twice in the calculation: once as a part of loop A, and the second time in loop B. Therefore, the nodal demands are:

$$Q_1 = (Q_{1-2} + Q_{1-4})/2$$
$$Q_2 = (Q_{1-2} + Q_{2-3} + Q_{2-5,A} + Q_{2-5,B})/2$$
$$Q_3 = (Q_{2-3} + Q_{3-6})/2$$
$$Q_4 = (Q_{1-4} + Q_{4-5})/2$$
$$Q_5 = (Q_{4-5} + Q_{5-6} + Q_{2-5,A} + Q_{2-5,B})/2$$
$$Q_6 = (Q_{5-6} + Q_{3-6})/2$$

For larger systems, a spreadsheet calculation is recommended. Alternatively, the same approach can be tried directly from the map by allocating a portion of the area of each loop to a corresponding node and determining the flow to be supplied from that node.

From Figure 4.27:

$$Q_1 = Q_4 = 0.25Q_A$$
$$Q_2 = Q_5 = 0.25(Q_A + Q_B)$$
$$Q_3 = Q_6 = 0.25Q_B$$

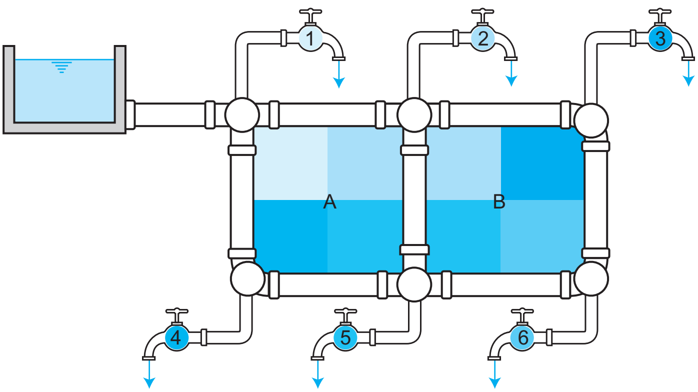


Figure 4.27. Nodal demands – graphic approach.

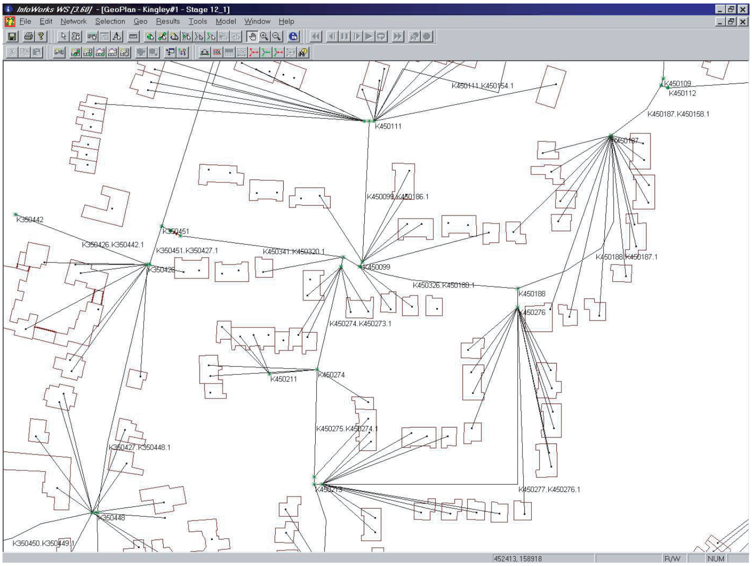


Figure 4.28. Info Works WS – demand allocation (Source: Wallingford Software Ltd).

The earlier method is a simplification of reality, and good enough as an initial guess. Throughout the process of model calibration, the nodal demands calculated in this way need to be adjusted because of the impact of major consumers, large buildings, uneven leakage distribution, etc. Good monitoring of the system as well as consistent billing records give crucial support in this situation. In more powerful programmes, this information can be directly allocated to the appropriate node, when the precise location of the house connection is known. An example is shown in Figure 4.28.

Self-study:
Spreadsheet lesson A5.8.11 (Appendix 5)

4.3.5 *Model testing*

Model validation and calibration

Once the first simulation run has been completed, the immediate concern is whether the results match reality. In this phase, several runs have to be executed which must confirm that:

– the model has a logical response to the altering of the input data; the simulation runs are in this case functioning in the *model validation*,
– the model is behaving in relation to the real system; comparison of the calculation results with the hydraulic measurements is part of the *model calibration*.

There can be different reasons why the earlier conditions are not satisfied. The input file can be accepted by the programme as correct in syntax, but:

- Some input data were (badly) estimated, because the real values were not known.
- The network was transferred to the model with some typing errors or data was omitted.
- The format of the input file was incorrect but the error was not (clearly) defined in the error library: e.g. too high calculation accuracy, insufficient maximum number of iterations, impossible operation mode specified, etc.
- The field measurements used for the model calibration were inaccurate.

Computer models cannot totally match a real situation; the results should always be judged based on the quality of input data and the measurements used for model calibration.

4.3.6 *Problem analysis*

With the correct execution of all the previous steps, the analysis of the problem is the final step of the modelling process and probably the shortest one. Some of the typical problems that can be solved by the help of a computer model are:

1. The selection of optimal pipe diameters for a given layout and demand scenario.
2. The selection of optimal models for pumps.
3. The selection of optimal position, elevation and volume of the reservoir(s).
4. The optimisation of the pump scheduling (to minimise energy consumption).
5. The optimisation of the reservoir operation (water depth variation).
6. The optimisation of the valve operation.
7. The simulation of fires.
8. The planning of pipe flushing in the system.
9. The analysis of failures of the main system components (risk assessment).
10. The analysis of water quality in the system (chlorine residuals, water age and mixing of water from various sources).

In many of these problems, the advantage of a quick calculation combined with proper analysis of the model response to the change of input data will lead to correct conclusions on the network performance after a series of ‘trial and error’ simulations.

The new generation of computer programmes based on optimisation algorithms (genetic algorithms and neural networks) tries to shorten the analysis even further. During the simulation process, the programme will try to satisfy a number of optimisation criteria set by the user. A typical example of optimisation problem is the analysis of the least cost pipe maintenance, in which the programme recommends the most economical measure from the list based on the network condition, the network performance and the unit cost of particular maintenance measure. Despite recent breakthroughs, these programmes are still in the development stage and are yet to match the size of network and calculation times achievable by using traditional models.

There is a wide range of literature on the subject of water distribution network modelling. A very comprehensive overview of the methods and applications can be found in Walski *et al*. (Haestad Methods, 2003).

4.4 HYDRAULIC DESIGN OF SMALL PIPES

Computer models are rarely applied while designing systems in small residential areas and/or pipes of indoor installations; standardisation is a more popular approach for small (service) pipes rather than attempting to size them precisely. This ensures adequate system flows and pressures under ordinary supply conditions, avoiding detailed hydraulic analyses. Moreover, fire flows can often be a dominant component of design flows, over-riding the peak flows caused by a relatively small number of consumers. Hence, adopting unique diameters makes maintenance easier and initially allows some buffer capacity for irregular situations. Some of the design methods are discussed further.

4.4.1 *Equivalence Method*

The Equivalence Method relates the design diameter of a pipe to the number of consumers that can be served by it. The equivalence table has to be prepared for each specific situation, based on the following input data:
– specific (average) consumption,
– a simultaneity diagram,
– the design velocity or hydraulic gradient.

The calculation is normally carried out for a number of available (standard) diameters. A sample equivalence table is shown in Table 4.4 based on specific consumption of 170 l/c/d, the simultaneity curve from Figure 2.7 and velocity of 0.5 m/s.

The design capacity Q in the table is calculated based on the design velocity or the hydraulic gradient. For instance, in the case of pipe $D = 60$ mm:

$$Q_{60} = \frac{D^2\pi}{4}v = \frac{0.06^2 \times 3.14}{4} \times 0.5 \times 3600 \approx 5 \text{ m}^3/\text{h}$$

Table 4.4. Equivalence table (170 l/c/d, 0.5 m/s).

D(mm)	Q(m^3/h)	pf_{ins}	Consumers
60	5	42	17
80	9	28	45
100	14	21	95
150	32	13	350
200	57	9	920
250	88	7	1900
300	127	5	3500

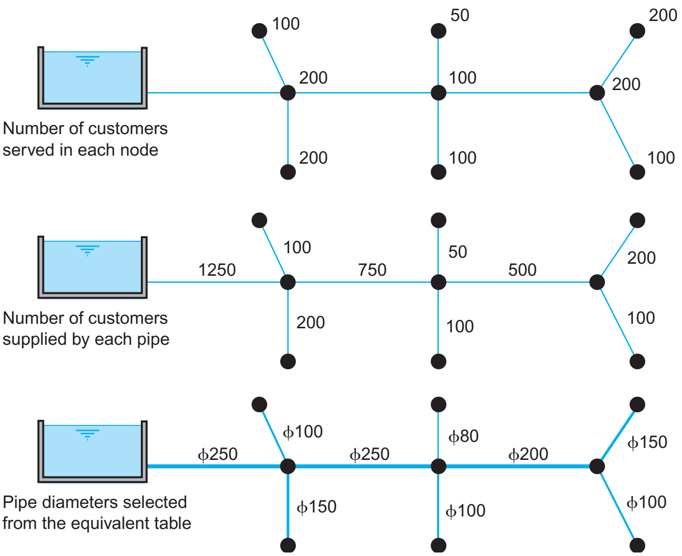


Figure 4.29. Example of pipe design using the Equivalence Method.

This flow has then to be compared with the peak supply condition, which is determined from the specific average consumption and the instantaneous peak factor for a corresponding number of consumers. In the earlier example:

$$Q_{60} = pf_{\text{ins}} n_i q_i = \frac{42 \times 17 \times 170}{1000 \times 24} \approx 5 \text{ m}^3/\text{h}$$

Correlation with the number of consumers may require a few trials before the results of the earlier two calculations match. Once the table is complete, a defined number of customers supplied from each pipe can be directly converted to the design parameter. An illustration of the principle for a simple system is shown in Figure 4.29 by using Table 4.4.

The Equivalence Method is used predominantly for the design of branched networks in localised distribution areas, from a few 100 up to

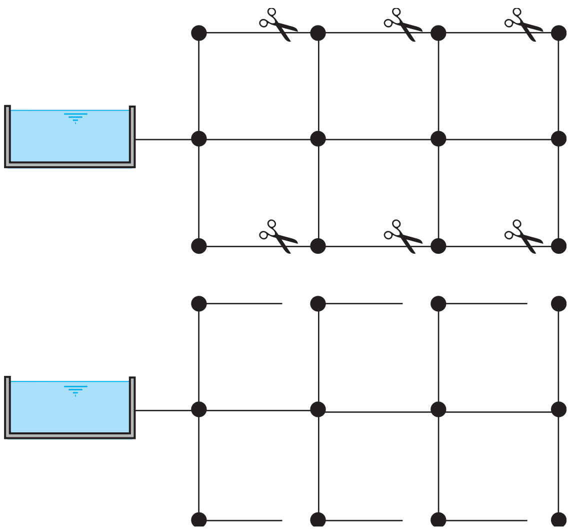


Figure 4.30. Conversion of the looped system into a branched system.

a few 1000 consumers. Simple looped systems can also be designed with this method after converting the grid into an imaginary branched type system. This can be done in several ways, most usually by 'cutting' the connections of the pipes expected to carry low flows (see Figure 4.30).

Practical difficulties in applying the Equivalence Method lay in the simultaneity diagrams, which are often difficult to generate because of the lack of reliable data. Creating an accurate instantaneous diagram requires good monitoring of the network with a rather long history of measurements. Alternatively, the diameters of the small pipes can be sized, based on the statistical analysis of the peak flows.

4.4.2 *Statistical methods*

Statistical methods are predominantly used for the design of service connections and indoor pipes. They can also be an alternative to the Equivalence Method when the simultaneity diagram is not available.

The design diameter is directly related by this approach to the peak capacities, which were determined based on the locally established standards for indoor outlets. Some of the statistical formulas for instantaneous demand calculation are presented later in the text.

In The Netherlands, the $q\sqrt{n}$*-method* offers an estimation of the peak instantaneous demand flow for a number of dwellings. The equation used is:

$$Q_{\text{ins}} = \sqrt{\sum_{i=1}^{r}\sum_{j=1}^{s} q_{i,j}^2} \qquad (4.18)$$

Table 4.5. Tap units, The Netherlands (VEWIN).

Draw-off point	n_i
Toilet	0.25
Toilet sink	0.25
Bathroom sink	1
Shower	1
Bath tap	4
Washing machine	4
Kitchen tap	4

where Q_{ins} is the peak instantaneous flow required by r dwellings, r is number of dwellings in the building, s is number of water outlets in one dwelling and q is standardised peak flow of each outlet.

To enable easy calculation, the maximum capacity of various types of outlets is expressed in so-called *tap units*. Equation 4.18 can then be transformed into:

$$Q_{\text{ins}} = q\sqrt{\sum_{i=1}^{r}\sum_{j=1}^{s}\left(\frac{q_{i,j}}{q}\right)^2} = q\sqrt{\sum_{i=1}^{r}\sum_{j=1}^{s} n_{i,j}} \tag{4.19}$$

where n is the number of tap units per outlet. This is a standard value, listed in Table 4.5 for $q = 300$ l/h.

A typical set-up of a family house in The Netherlands consists of one kitchen tap, one toilet with sink and a bathroom with another toilet, sink bath tap and the washing machine connection. The peak instantaneous flow in this case will be:

$$Q_{\text{ins}} = \frac{300}{3600} \times \sqrt{4 + (0.25 + 0.25) + (0.25 + 1 + 4 + 4)} = 0.31 \text{ l/s}$$

A similar approach is used in other European countries (Germany, Spain, Denmark, etc.).

4.5 ENGINEERING DESIGN

Engineering design deals with non-hydraulic aspects of water transport and distribution systems. It is based on technical and financial grounds and tends to standardise regarding the choice of components, materials, typical designs and installation or construction procedures.

Decisions such as whether to opt for a limited choice of the most suitable pipe materials, a number of typical pipe diameters, pumping units supplied by the same manufacturer and standardised valves, taps, bends, etc. that fit well with their dimensions, are of extreme importance as this obviously contributes to easier maintenance of the system and affects the value and volume of the stock. Water supply companies commonly adopt a material policy based on local conditions (costs, manufacturing, competitive supplies, delivery time, service, etc.) leading to standardisation in the entire supply area. In addition to that, standard designs for functional components of the water supply system, public taps, fire hydrants, house and yard connections, indoor installations, etc. need to be developed. In all of these activities, cooperation with respective local industries would normally provide a higher degree of self-reliance.

Finally, typical designs of pumping stations, elevated tanks, etc. can be proposed leading to a stage in which all these elements will be assembled into a kind of 'ready made' project, allowing short preparation periods, accurate cost estimations and uniformity. The feedback from experience in a number of such projects leads directly to improvements of the main design concept. The advantages for operational management and training purposes are also obvious.

For example, the experience in the use of pipe materials in The Netherlands shows interesting trends (Figure 4.31). The most widely used material in the first half of the twentieth century was cast iron (CI). In the sixties and seventies, asbestos cement pipes (AC) were massively introduced being corrosion free, with thinner walls and therefore lighter than the cast iron pipes. In the eighties and nineties, the dominant pipe material became polyvinyl chloride (PVC) being also corrosion free, more flexible, lighter and cheaper than AC and still strong enough owing to enhanced production technology, even in larger diameters.

As a result, the entire generation of CI pipes in the ground are nowadays some 60–80 years old, the AC pipes are mostly 30–50 years old,

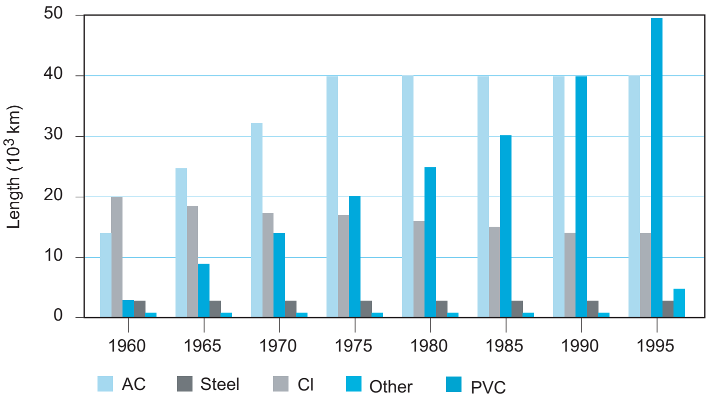


Figure 4.31. Development of pipe materials in The Netherlands in the period 1960–1995 (VEWIN).

while the PVC pipes are usually younger than 20–30 years. Such a picture gives a clear idea as to when the moment for massive renovation of the networks is going to occur. Consequently, the replacement of the CI pipes has been carried out to a large extent; it is expected that all AC pipes will be removed from the ground within the next 20–30 years, whilst that moment could be further delayed in the case of the PVC pipes.

Current preference for flexible, plastic based pipes is very logical for The Netherlands, given the predominantly unstable and corrosive soil conditions and flat topography where pressures above 60 mwc are not common in distribution networks. On the other hand, trends in some other countries may be entirely different. In neighbouring Germany for instance, ductile iron pipes are more frequently used in many areas than PVC pipes, owing to different topographic and soil conditions.

4.5.1 *Pipe materials*

While deciding on the most suitable pipe materials, the following pipe properties should be taken into account besides the conveying capacity (Brandon, 1984):
– maximum and minimum depth of pipe cover,
– details of backfill material,
– anticipated loading on ground surface,
– length and weight for handling and storage,
– resistance to corrosion and chemical action,
– permissible longitudinal and diametric deflection,
– pipe embedment and support conditions,
– ease of making repairs and future connections,
– ring stress to withstand heavy backfill loads without pipe deformation,
– in waterlogged ground, the weight of pipe in relation to the risk of flotation,
– pipe flexibility to be laid in a curved trench,
– pipe length with respect to the number of joints required,
– risk of damage from third parties,
– deformations caused by extreme temperatures, uneven subsidence, vegetation roots, etc.

A proper choice helps to avoid the following problems in the operational management of transport and distribution systems:
– frequent interruptions of supply,
– increased water and energy losses,
– deterioration of water quality,
– shorter pipe lifetime,
– expensive maintenance of the system.

Pipes used in water supply are made of various materials and, depending on their resistance to the backfill and shock loads, they can be categorised in three large groups:
– rigid: cast iron (CI), asbestos cement (AC), concrete,
– semi-rigid: ductile iron (DI), steel and
– flexible: polyvinyl chloride (PVC), polyethylene (PE), glass reinforced plastic (GRP).

Furthermore the pipe materials can be classified as metallic and non-metallic. Each of the pipe materials has a specific composition that determines its properties. An ideal material that can be applied in all conditions does not exist. The main properties of the most commonly used materials according to the US experience (Smith *et al.*, 2000) are compared in Tables 4.6 and 4.7. Some of those materials are elaborated further in this paragraph.

Cast Iron (CI) is one of the oldest pipe materials used for the conveyance of water under pressure. The use of cast iron pipes has declined over the last few decades. Nevertheless, it is not uncommon to come across a cast iron pipe that is 70–80 years old and still in good condition; considerable lengths exist in some networks even in developed countries, such as in the UK or USA. Two examples of old CI pipes shown in Figure 4.32 are from The Netherlands.

The main disadvantage of CI pipes is generally low resistance to external and internal corrosion. This reduces the pipe capacity and iron

Table 4.6. Properties of metallic materials in use in US (Smith *et al.*, 2000).

Characteristic	CI	Lined DI	Steel	Galvanised Steel	Copper Tubes
Internal corrosion resistance	Poor	Good	Poor	Fair	Fair
External corrosion resistance	Fair	Moderate	Poor	Fair	Fair
Cost	Moderate	Moderate	Moderate	Moderate	Moderate
Specific weight	High	High	High	Moderate	High
Life expectancy	High	High	High	High	High
Primary use	T/D[1]	T/D	T/D	T/D	S
Tapping characteristics	Fair	Good	Good	Good	Good
Internal roughness	Moderate to High	Low	Moderate to High	Moderate to High	Low
Effect on water quality	High	Low	Moderate	Moderate	Moderate to High
Equipment needs	Moderate	High	Moderate	Moderate	Low
Ease of installation	Low to Moderate	Low to Moderate	Low to Moderate	Low to Moderate	Moderate
Joint water-tightness	Fair	Very good	Very good	Fair	Good
Pressure range (mwc)	NA	100–250	Varies	NA	Varies
Diameter range (mm)	NA	80–1600	100–3000	NA	<50
Ease of detection	Good	Good	Good	Good	Good

[1] T-transport, D-distribution, S-service connections.

Table 4.7. Properties of non-metallic materials in use in US (Smith *et al*., 2000).

Characteristic	AC	Reinforced concrete	PVC	PE	GRP
Material category	Concrete	Concrete	Plastic	Plastic	Composite
Internal corrosion resistance	Good	Good	Good	Good	Good
External corrosion resistance	Good	Good	Very good	Very good	Good
Cost	Low	Moderate	Low	Low	High
Specific weight	Moderate	Moderate	Low	Low	Low
Life expectancy	Moderate	High	Moderate	Moderate	High
Primary use	D[1]	T	D	S/D	Storage
Tapping characteristics	Fair	Fair	Poor	NA	NA
Internal roughness	Low to Moderate	Low	Low	Low	Low
Effect on water quality	Low	Low to Moderate	Moderate	Low	Low
Equipment needs	Moderate	High	Low	Low	Moderate
Ease of installation	Moderate	Low to Moderate	Moderate to High	High	Low to Moderate
Joint water-tightness	Good	Good	Good	Poor	NA
Pressure range (mwc)	70–140	Max. 160	Max. 160	Max. 140	NA
Diameter range (mm)	100–1100	300–4000	100–900	100–1600	NA
Ease of detection	Poor	Fair	Poor	Poor	Poor

[1] T-transport, D-distribution, S-service connections.

Figure 4.32. Old cast iron pipes.

release causes the water quality to deteriorate. Although the corrosion levels could stabilise after some time, the problems are usually solved by applying an internal cement lining and/or an external bituminous coating. The considerable weight and required wall thickness are additional disadvantages of these pipes.

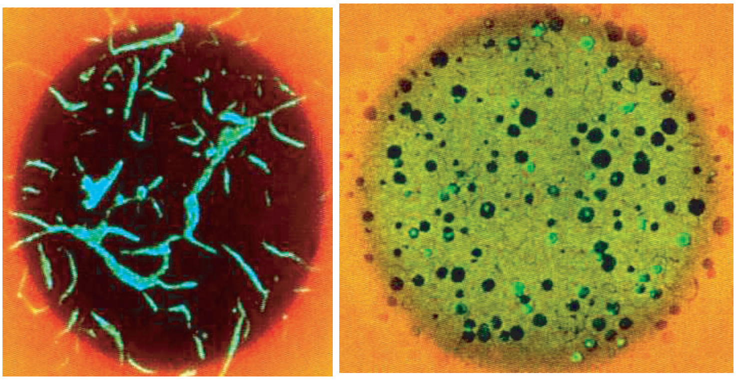


Figure 4.33. Texture of the grey iron and ductile iron materials (Saint-Gobain).

Ductile iron pipes

DI is a material that has been in use for almost 60 years. It is an alloy of iron, carbon, silicon, with traces of manganese, sulphur and phosphorus. Unlike ordinary CI pipes made of grey iron where free carbon is present in flakes, the carbon in DI pipes is present in the form of discrete nodules (see Figure 4.33), which increases the material strength by 2–2.5 times.

DI pipes are strong, durable and smooth pipes, usually laid in mid-range diameters (100–600 mm) although they can be manufactured up to 1800 mm. They are suitable for almost all soil conditions if corrosion protected. Compared to the CI pipes, DI pipes are lighter due to reduced wall thickness (see Figure 4.34) as well as being less susceptible to external loadings.

Despite their reduced weight, these pipes are still rather heavy and handling with appropriate machinery is necessary even in smaller diameters. Figure 4.35 shows large DI pipes positioned alongside the trench in which they shall be laid. In aggressive soil conditions, DI may become susceptible to corrosion. As a minimum, bituminous paint should be used as an external protection. An internal cement lining is regularly applied to prevent corrosion resulting in turbidity and colour that may appear in water.

Steel pipes

Steel pipes are manufactured in two ways: by welding steel plates or stripes either longitudinally or in the form of a spiral (large diameters), or seamless from a steel billet (small diameters). These alternative manufacturing processes cover a wide range of diameters, wall thickness and fittings that can be applied to a variety of network layouts and working pressures. Typical diameters are between 100 and 1800 mm but more frequently these pipes are used for the transport of large water quantities. Steel pipes are the ultimate choice for piping in pumping stations.

Compared to iron or pre-stressed concrete pipes of the same internal diameter, steel pipes are stronger, more flexible and have thinner walls. Consequently, they are lighter and easier for handling and laying.

Figure 4.34. Cement lined DI pipe with PE coating.

Figure 4.35. DI pipe length.

Moreover, they are available in longer units, which reduces the total number of joints required (see Figure 4.36).

Second, repair of the pipes is relatively easy and can be conducted under space restriction. Leaks would normally be localised to joints or pinholes resulting in an acceptable loss of water. An additional safeguard in this respect is to weld the joints.

Figure 4.36. Cement lined steel pipe with PE coating.

Just as with other metal pipes, steel pipes are sensitive to corrosion. Hence, the internal and external protection must be perfect for both pipes and joints; an example of triple layer coating is shown in Figure 4.37.

Copper pipes

A large number of service pipes and plumbing inside premises is made of copper. This material is popular as it is relatively cheap and reliable for large-scale implementation. As they are not used for distribution pipes, copper tubes rarely exceed 50 mm in diameter. The material is strong and durable, whilst at the same time sufficiently flexible to create any kind of bend as shown in Figure 4.38. Low internal roughness helps to minimise the hydraulic losses resulting from long pipe lengths.

Asbestos cement pipes

AC pipes are rigid non-metallic pipes produced from a mixture of asbestos fibre, sand and cement. The carcinogenic effect of asbestos-based

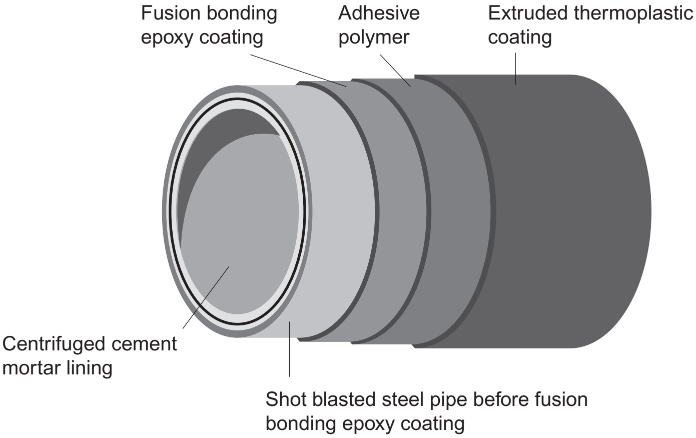


Figure 4.37. Steel pipe coating (Manufacturer: Europipe).

Figure 4.38. Indoor installations made of copper.

materials used in water distribution has been studied carefully in the last couple of decades. Although not dangerous when in drinking water, the fibres can be harmful when inhaled. Therefore, the laying of new AC pipes has been prohibited by law in many countries, due to possible hazards during manufacturing, maintenance and disposal of these pipes.

The pipes that are still in the ground are typically of smaller diameters although larger diameters can also be found. The main advantages of the AC pipes, compared to iron pipes, are:

– freedom from internal corrosion,
– generally better resistance to soil corrosion,
– a smooth inner surface,

– lighter weight and
– lower production costs.

The pipes can be drilled and tapped for service connections but are not as good as iron pipes in this respect; these locations are potential sources of leakages. Bursts and longitudinal deformations are also more likely and AC pipes in aggressive soils tend to corrode.

In The Netherlands, the manufacturing of AC pipes was stopped in 1993. The pipes still remaining in operation are handled with the utmost care; if they malfunction, the pipe will be replaced by a new one, mostly made of PVC or PE.

Concrete pipes

Concrete pipes (Figure 4.39) are rigid, cement-based pipes mainly used for sewerage. In drinking water supply, they will be more frequently laid for water transport than distribution. They are produced in diameters of between 250 and 1600 mm. They can occasionally be even larger, in which case they are almost always reinforced and usually pre-stressed to withstand internal pressure and external loads. Pre-stressed concrete pipes can be reinforced in two ways: circumferentially pre-stressed with a steel cylinder, or both longitudinally and circumferentially pre-stressed with steel net.

Figure 4.39. Old concrete pipes reinforced with steel net.

As a material, concrete is lighter than iron or steel but concrete pipes are generally heavier than the metal pipes of corresponding inner diameter, owing to a much thicker wall. For example, a pipe of $D = 600$ mm has a mass of some 500 kg/m length, which creates difficulties during transportation and handling. The advantage of this heavy weight is that it limits any risk of movement, specifically in waterlogged ground. Furthermore, concrete pipes can carry heavy loads without damage or deformation and show good corrosion resistance; no special precaution is needed for pipe bedding or backfilling. Low internal roughness enables good hydraulic performance while conveying large water quantities. Last but not least, concrete pipes are comparatively cheap in large diameters.

Laying a route with concrete pipes requires more time than with metal pipes and involves heavy machinery. Operationally, a pipe repair also takes longer than with other types. If in the case of failure an alternative supply is not available, a larger part of the distribution area may be excluded from service. As with AC pipes, aggressive soil or water can cause corrosion that is normally restricted to cement mortar but will attack the steel reinforcement once it is exposed to the environment; external coating is needed in such cases.

Figure 4.40 shows the 80-tonne pipe, $D = 4000$ mm, $L = 7.5$ m made of pre-stressed concrete, used in the 'Great Man Made River' water transportation project in Libya.

Polyvinyl chloride pipes

PVC pipes are flexible pipes widely used in a range of diameters up to 600 mm (see Figure 4.41). The properties of this thermoplastic material give the following advantages:
– excellent corrosion characteristics,
– light weight,
– availability in long pieces,

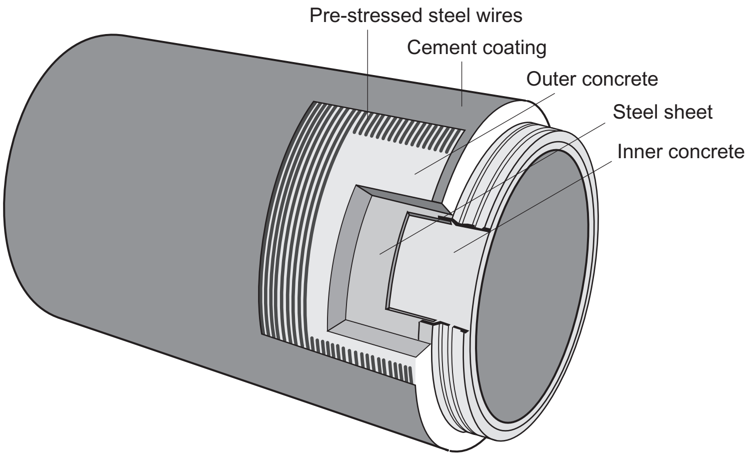


Figure 4.40. Pre-stressed concrete pipe in the 'Great Man Made River' project in Libya (The Management and Implementation Authority of the GMR Project, 1989).

Table 4.8. Reduction in pressure rating of PVC pipes (Mays, 2000).

Service temperature (°C)	27	32	38	43	49	54	60
Percentage of original pressure rating	88	75	62	50	40	30	22

Figure 4.41. PVC pipes in larger diameters.

– low production costs,
– reduced installation costs.

The disadvantages lie in the reduction of its impact strength in extremely low temperatures. An incidence of pipe bursts in wintertime has direct relation to the depth of the trench, which will be adopted based on the comparison between the costs of excavation, pipe repairs and estimated water losses. Furthermore, the PVC pipes lose their tensile strength in extremely high temperatures. Table 4.8 shows the reduction of the pressure rating for the PVC pipes depending on the operating temperature (Mays, 2000). As a result, careful handling, stacking and laying is crucial under extreme temperatures. Pipe support must be as uniform as possible, free from stones or other hard objects, and the filling material used must be well compacted.

Despite radical improvement in the manufacturing process, PVC pipes are uncommon in diameters above 600 mm. Larger diameters rarely withstand pressures much above 100 mwc and are therefore less suitable for high-pressure flows, e.g. pressure pipes in pumping stations, or trunk mains. Moreover, a fracture can develop into a split along considerable lengths of the pipe, resulting in large water losses.

Pipe permeability

Another problem in operation occurs when the pipe is exposed to organic soil pollutants (oil, gasoline, etc.) over long periods. Even in constantly pressurised pipes and without leakage, water quality may be affected after several months. This is a consequence of organic molecules passing through the pipe wall, which is known as *pipe permeability*. The result of it is taste and odour problems with considerable health hazards. In addition, the pipe material will be softening, which weakens its structural strength. Laying of PVC pipes is therefore not advised in the vicinity of refineries or petrol stations.

Polyethylene pipes

Another polymer used for pipe production is PE. There are three phases in the development of the manufacturing technology resulting in the following types of PE pipes:
– low density PE (LDPE), manufactured previously exclusively for service connections in diameters $<$50 mm,
– medium density PE (MDPE), with improved performance and for diameters up to 200 mm,
– high density PE (HDPE), nowadays also manufactured in large diameters (exceptionally above 800 mm).

Compared to PVC, PE pipes show the following enhanced characteristics:
– improved resistance to stress cracking,
– better performance under extreme temperatures,
– extreme flexibility,
– good welding compatibility,
– improved resistance to surge pressures.

Furthermore, handling of the pipes is easy. Smaller diameters (up to 100 mm) can be rolled in coils up to 150 m (Figure 4.42), while larger pipes are 10–12 m long.

One of the main disadvantages of PE compared to PVC is the higher price resulting from the thicker walls required for these pipes.

Figure 4.42. Stock of coiled PE pipes.

Permeability of PE is also an issue. Maintenance problems mostly relate to jointing of the pipes. The welding technique is reliable but involves qualified personnel and electrical equipment. Also, special precautions have to be taken when using conventional mechanical fittings, due to the creeping of polyethylene. Water quality problems related to this material result from bio-film formation on the pipe wall.

Glass reinforced plastic pipes

GRP pipes are composed of three main components: fibreglass, resin and sand. The strength of the pipe is derived from bonding the fibreglass with resin. The purpose of adding sand is the increase of wall thickness that improves stiffness of the pipe, which makes its handling easier.

GRP pipes are commonly manufactured in larger diameters (see Figure 4.43) and thus used for water transport. Compared with concrete and steel pipes of the same size, they are:

– lighter in weight,
– more flexible,
– more resistant to corrosion.

In relation to the technical lifetime of pipes, GRP is still a relatively new material as it has only been in use since the eighties. Therefore there is insufficient experience of its long-lasting performance to be able to draw firm conclusions. It is rather an expensive material with excellent corrosion characteristics, which makes it predominantly suitable for industrial and chemical sites where intensive corrosive conditions may occur. Applications of this material frequently include the transport of waste

Figure 4.43. GRP pipes and fittings.

water. Other applications in drinking water supply include the lining of storage tanks.

Efforts have been made in the last couple of decades to judge pipe materials not only from the perspective of their performance in use but also on the effects they cause on the environment either during manufacturing or in the post-exploitation phase. A typical example is PVC, which is considered to be a favourable pipe material but which emits some carcinogenic pollutants during manufacturing processes.

Although it is difficult to draw general conclusions due to the variety of criteria that have to be analysed, the assessment may be based on the following questions (van den Hoven *et al*., 1993):

– How much of the natural reserves of raw materials that are used for pipe production are exhausted?
– How high is the energy input in production?
– What is the amount and structure of solid wastes produced in the manufacturing processes?
– Are there hazardous substances released to the environment in the production phase?
– Can the pipe be re-used (re-cycled) after it has been exploited?

According to van den Hoven *et al*. the comparison between CI-, steel-, AC-, PVC- and GRP pipes lead to the following conclusions:

– The environmental risks of exhaustion of natural reserves are acceptable in the case of the materials mentioned, with the exception of CI pipes when a zinc coating is applied. The effect can be reduced if a PE coating is used instead.
– Energy input in the pipe production is severe in the case of the CI- and steel pipes (35 to 40 $\times$ 10^9 J per 100 m of the pipe length with a diameter of 100 mm). The figure for other pipe materials is 3–6 times lower, which is assumed to be reasonable.
– Emission of toxic substances, as shown in Table 4.9, can be observed during production.

CI-, steel- and PVC pipes score better than other materials in the post-exploitation period. The CI and steel can be completely re-used but zinc- or PE coating has to be renewed. PVC can be re-used 6–10 times resulting in slightly lower quality every new cycle. Re-cycling the GRP pipes saves around 30% of new raw material. AC remains dangerous because of the carcinogenic effect of AC fibres also in this phase.

As for the evaluation of their characteristics that is relevant for operation, any classification of pipe materials on an environmental basis is also difficult. AC is recognised as a potentially highly hazardous material but all the others also have adverse consequences in one or other phase. Pipe production is the most critical aspect in this respect. Despite its limitations, this kind of analysis is obviously useful in pointing out

Table 4.9. Emission of toxic substances as a result of pipe production (van den Hoven and van Eekeren, 1988).

Pipe production	Emission into air	Pollution of water	Solid waste
CI, steel	CO_2 SO_2 NO_x	Sulphur Ammonium Phenol Cyanide Fluoride Lead Zinc Chloride	Heavy metals
AC	AC fibres		
PVC	SO_2 NO_x Vinyl Chloride		Vinyl Chloride
GRP	Heavy metals Acids	Hydrocarbon	

critical manufacturing steps, which should be improved for minimising the environmental risks. In addition, it initiates research oriented towards possible new materials (e.g. glass reinforced AC that might prevent the deficiencies of standard AC).

4.5.2 *Joints*

As with pipes, the joints that connect them can be classified as rigid, semi-rigid and flexible. For instance, flexibility in the routing of rigid pipes can be improved if flexible joints are used to connect them. On the other hand, some flexible pipes are usually welded to provide safe water-tight connection.

Standardisation of joints does not really exist. It is wise to limit the choice of joints to a few types; mixing different manufacturers and models may create stocking and repair difficulties. Finally, poor jointing is often a major source of leakage. Hence, special attention should be paid to provide water tightness and protection from corrosion.

Welded joints

Welded joints are of a rigid type, suitable for steel and PE pipes (see Figure 4.44). This is the cheapest joint for steel pipes of larger diameters, strong enough to carry high water pressure and longitudinal strain. These joints do not allow any pipe route deflection and change in direction should be provided by proper fittings.

Electro-fusion

Welding of PE pipes involves a process called *electro-fusion*; the ends of two consecutive pipes are melted by a heated plate, and then coupled at

Figure 4.44. Welding of steel pipes.

Figure 4.45. Welding and laying of PE pipes.

extreme pressure (approximately 70–80 bar) for about 25–30 minutes. This is a very convenient method because the jointing can be done before the pipes are laid, which enables the creation of long sections very quickly (Figure 4.45). As a result, narrow trenches can be adopted, which reduces the costs of excavation. Moreover, PE pipes of different density can be connected by this method.

Flanged joints

Flanged joints (Figure 4.46) are predominantly applied for rigid pipes, wherever a need for temporary disconnection occurs (pumping stations, cross-connection chambers, etc). They can be an integral part of the pipe, screwed on or welded to it. Connection bolts allow the work to be executed quickly while the rubber ring inserted between the flanges enables a water-tight connection.

The flanged joints are mostly applied to metal pipes but they are also available with PE pipes, making the connection of these two materials possible. The example from Figure 4.46 shows the jointing of PE- and DI materials. Corrosion of the bolts may be a concern if the pipe is laid in aggressive soil.

Spigot and socket joints

Spigot and socket joints are flexible joints that allow a deflection between connected pipes. The traditional caulked lead spigot and

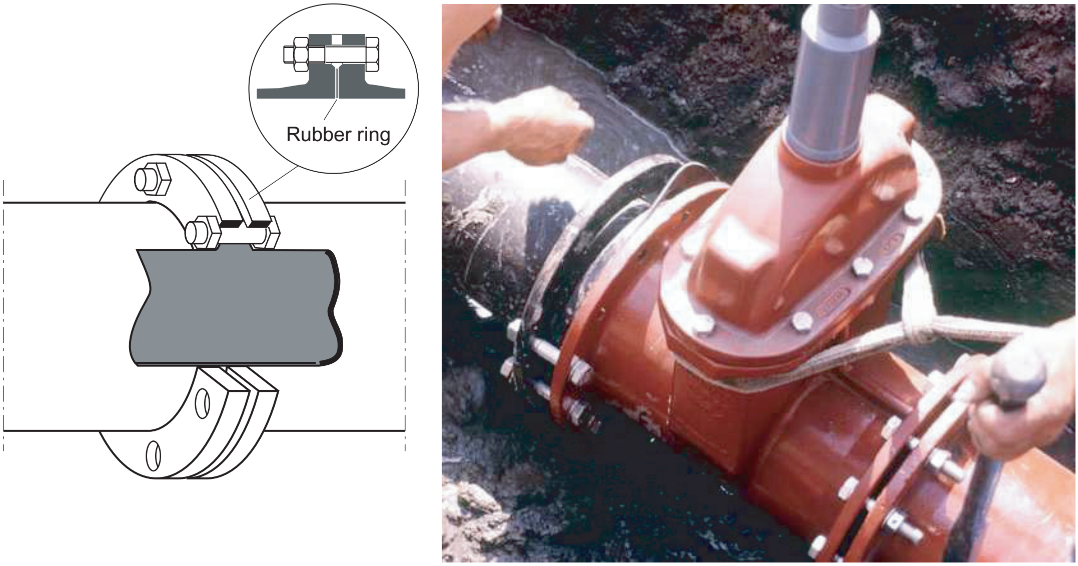


Figure 4.46. Flanged joints.

socket joint was extensively implemented on CI pipes in the past. Nowadays, this type has been replaced by a variety of models that are also applicable for most other materials. The differences in the connection are mainly based on the filling of the space between the spigot and socket.

Sealing ring joints

Sealing ring joints (Figure 4.47a) are made by a rubber gasket that is forced into a sealing position by the spigot and socket. The jointing is obtained by pushing one pipe into another (also known as 'push-in' joints). The water pressure adds to the tightening of the connection so that additional mechanical compression is not necessary. This type of connection is suitable for pre-stressed concrete-, DI-, PVC- and GRP pipes. Deflection of the pipes is allowable from 1–5°, depending on pipe material. The joint strength can be improved by welding of the pipes but this reduces the deflection. Another alternative is to coat the joint with polyethylene after the connection has been made.

Gland joints

Gland joints are connections where the rubber ring is forced by a rigid peripheral gland into a space between the spigot of one pipe and the socket of the next (Figure 4.47b and 4.48 right). This kind of connection has no significant longitudinal strength but allows deflections of 4–6°. It is commonly used for CI- and DI pipes, but is also suitable when steel- and AC pipes are connected to DI pipes.

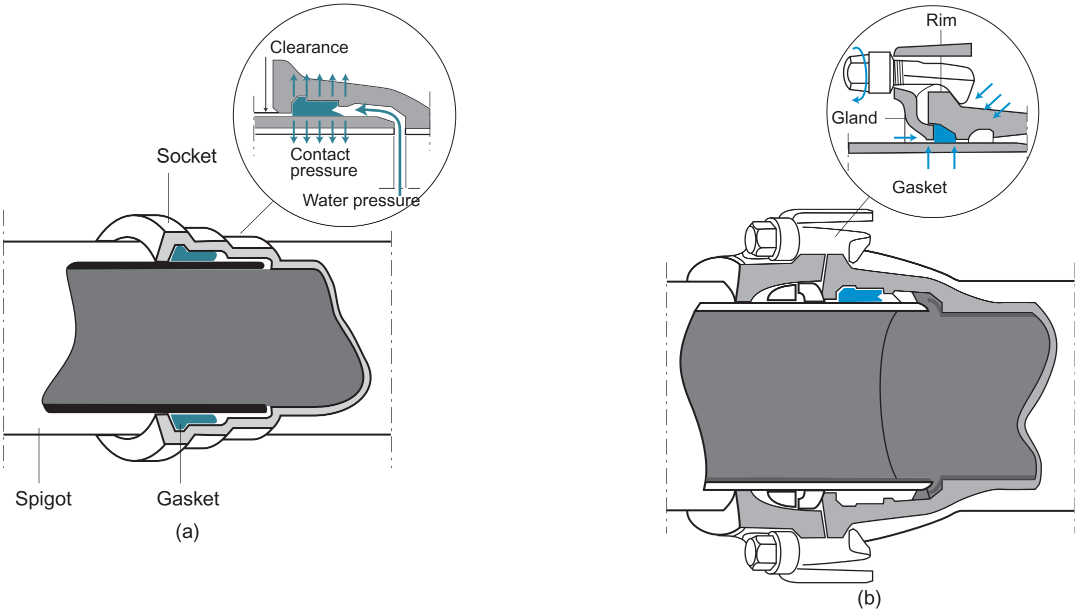


Figure 4.47. Spigot and socket and gland joints.

Figure 4.48. Push-in and gland joints (Manufacturer: Saint-Gobain).

4.5.3 *Fittings*

Fittings are applied in situations that require a change in pipe diameter and/or material, pipeline direction, or when valves, water meters or hydrants have to be installed. A variety of designs exist. Several models made of ductile iron are shown in Figure 4.49.

It is normal to select pipes and joints from the same manufacturer although mixing of materials is also possible. For example: steel fittings

Figure 4.49. DI fittings (Manufacturer: Saint-Gobain).

can be used with pipes made of other common materials, PVC may be combined with high density PE, and DI and PE can also be jointed.

The proper selection and installation of fittings is very important because they are, as with joints, very often a source of leakage. On most occasions information about the construction, dimensions, application range, installation and maintenance is available in the manufacturer's catalogues.

4.5.4 *Valves*

Valves in water distribution systems are distinguished by their principle of operation, the role in the system and the manner of control. Generally, the valves fulfil three main tasks:

– flow and/or pressure regulation (flow control valves, pressure reducing- or pressure sustaining valves, etc.),
– exclusion of parts of the network due to emergency or maintenance reasons (section valves)
– protection of the reservoirs and pumps (e.g. float valves, non-return valves).

Operation of the closing element causes the flow turbulence that create a pressure loss. The magnitude of this loss is defined by the relationship between the valve position and the head-loss factor (a form of Equation 3.25). Manufacturers commonly supply a diagram known as the *valve characteristics* for each specific valve type and diameter. An example of this was shown in Figure 3.13.

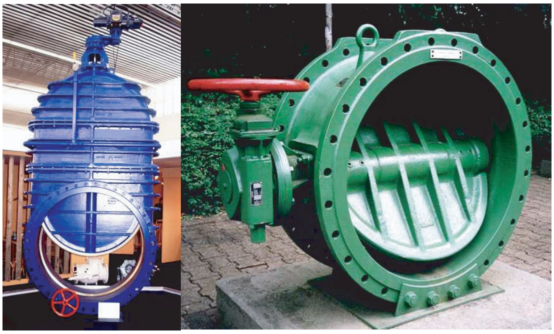


Figure 4.50. Large gate- and butterfly valves.

Figure 4.51. Blocks of gate valves.

Valve characteristics

Various valve constructions are based on the motion of the closing element, which can be linear (e.g. in case of gate or needle valves) rotation (butterfly valves) or deformation (membrane valves). Gate- and butterfly valves, as shown in Figure 4.50, are the most frequently used valves in water transport and distribution.

Gate valves

Gate valves function predominantly to isolate a pipe section; a valve block will be installed on an intersection between the pipes (Figure 4.51). Consequently, these valves would normally operate in an open/closed position. Flow regulation is possible but is not common; the disk that is partly exposed to the flow may eventually loosen, causing leakage when it is in the closed position.

The hydraulic performance of gate valves is good, as the disc is fully lifted in the open position reducing minor losses to a minimum; this is also useful if the pipe is cleaned mechanically. The disadvantage is, however, that the bonnet of the disc requires additional space around the valve.

As a prevention against surge pressures, the gate valves have to be continually open or closed for a long time, which makes them unsuitable in places where more frequent valve operation is required. It may sometimes take a half an hour before a large gate valve is brought from one extreme position to the other. The process becomes even more difficult during the opening, as the thrust force acts only at one side of the disc. A bypass with a smaller valve is therefore recommended in the case of larger valve diameters, which is used to fill the empty section with water and even the pressures on the disc.

Butterfly valves

Butterfly valves have the disc permanently located in the pipe, rotating around a horizontal or vertical axis. When the valve is fully open, the disc will be positioned in line with the flow, creating an obstruction that increases the head-loss compared to a fully open gate valve.

The butterfly valves are widely used in pumping stations as they are compact in size, easier to operate and cheaper than the comparable gate valves. They are also frequently applied in distribution networks, the main disadvantage being the obstruction created by the disc that makes mechanical cleaning of the pipe impossible. In both cases, the valve will be predominantly operated in an open/closed position but some degree of flow regulation is also possible. Nevertheless, using them for a high-pressure throttling over a longer period may damage the disc. Here as well, operating the valve operation too quickly is a potential source of surge pressures.

Valve regeneration

The number of section valves in any sizable distribution system can be huge, with the vast majority of them not being frequently operated. An automatic device with an adjustable turning speed can be used in cases where many valves have to be operated within a short period of time. Occasional turning of valves, known as *valve regeneration*, is a part of regular network maintenance in order to prevent clogging of the mechanism (Figure 4.52).

Valves in water transport and distribution systems can also be distinguished according to their role in the system.

Float valve

Float valves are common devices in preventing reservoir overflows. They are automatically controlled by the surface water level in the reservoir. The principle of operation is shown in Figure 4.53.

The closing element of a butterfly valve is connected to a floating body, or a number of sensors at different elevations. Starting from the preset level, progressive throttling of the valve will occur as the water level rises, until the top level is reached. In this position, the valve will become fully closed. At the water level below the critical level, the valve remains fully opened.

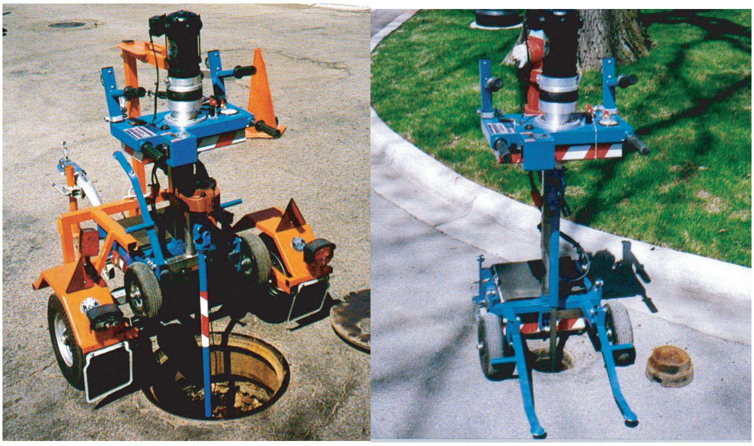


Figure 4.52. Valve turning machines (Manufacturer: Hydra-Stop).

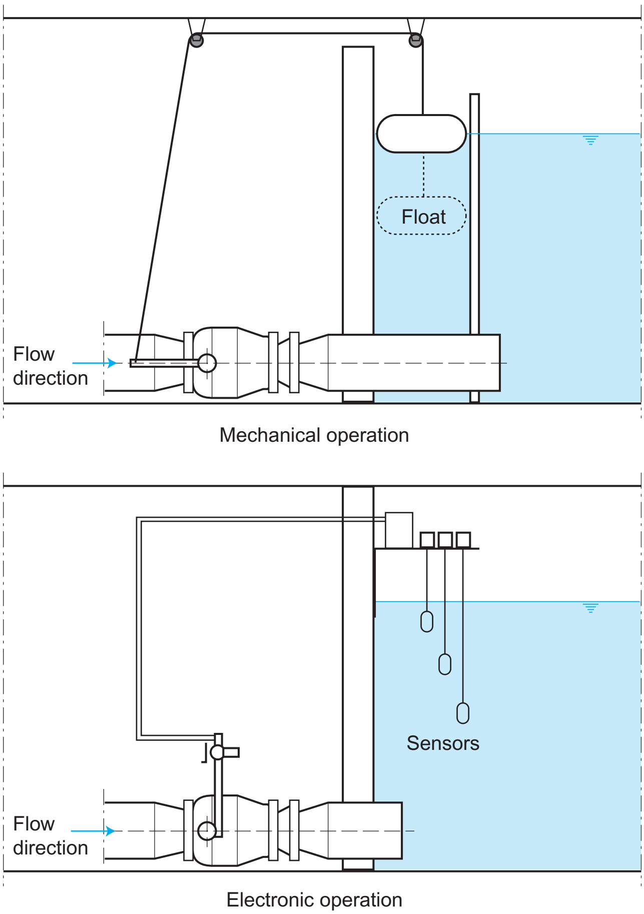


Figure 4.53. Operation of the float valve.

Figure 4.54. Large non-return valve: cross-section (left), and in operation (right).

Non-return valve

Non-return valves (also known as check-, retaining- or reflux valves) are the valves that allow flow in one direction only. An opposite flow direction causes the valve to close and remain closed until the flow is re-established in its original direction. Hence, these valves operate in an on/off position, either fully closed or opened by the flow itself.

The non-return valves are installed in pumping stations (discharge pipes) as back-flow prevention; an example is shown in Figure 4.54. Sometimes they may be found on distribution pipes or as a part of the service connection.

Pressure reducing valve

Pressure reducing valves are normally used to control the pressure in isolated parts of networks if it becomes too high. When the pressure upstream of the valve grows above the preset value, the valve will start closing until the downstream pressure is equal to the preset pressure. If the upstream pressure is below the preset value, the valve operates as fully opened.

Pressure reducing valves also operate as non-return valves: when the downstream pressure is higher than the upstream pressure, the valve is shut off. Consequently, these valves are equipped with upstream and downstream pressure gauges in order to maintain proper functioning (Figure 4.55).

Pressure sustaining valve

A *pressure sustaining valve* is in fact a pressure reducing valve in reversed operation. In this case the isolated section of the network is upstream of the valve, where a certain minimum pressure should be guaranteed. The valve starts closing if the upstream pressure falls below the preset value.

Air valve

An *air valve* is a special type of valve that helps to release air from pipelines, which prevents reduction of the conveying capacity. Air accumulation can occur during the filling of the pipeline but also in normal

Figure 4.55. Pressure reducing valve.

operation. The valve consists of a float arrangement contained in a small chamber with an orifice vent. When water is present in the chamber, the pressure that initiates up thrust of the float closes the orifice. The appearance of air in the chamber depresses the water level uncovering the orifice. The air is expelled until normal water pressure in the chamber is established again.

Air valves are distinguished by the different size of orifice diameters, the number of chambers (single, dual) and the operating pressure. An example of a dual chamber valve is displayed in Figure 4.56.

The right chamber functions as an ordinary air valve, whereas the left (larger) chamber is used during low-pressure conditions: filling/emptying of the pipe or in any situation where negative pressures may occur.

Finally, valves can be operated manually or automatically. Automatic operation is usually linked to a schedule (time controlled valves), pressure or water level (float valves, pressure controlled, pressure reducing or sustaining valves), flow rate or flow direction (flow controlled and non-return valves) somewhere in the system. A variety of mechanical, hydraulic and electronic equipment is nowadays involved in controlling valve operation, by sophisticated means that also allow remote operation.

The example in Figure 4.57 shows a butterfly valve that can be controlled both manually and electronically. The small window in the axis of the valve indicates the current position of the disc.

4.5.5 *Water meters*

The purpose of metering in water distribution systems is twofold: it provides information about the hydraulic behaviour of the network,

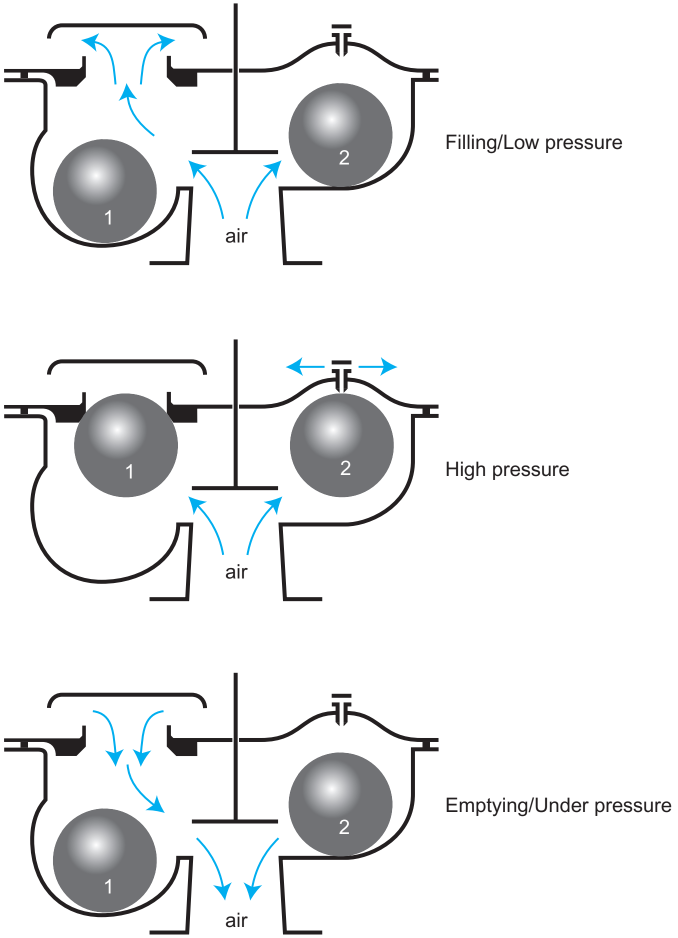


Figure 4.56. Operation principle of the dual-chamber air valve.

Figure 4.57. Manual and automatic butterfly valve control.

which is useful for the operation, maintenance and future design of the network extensions, as well as being a basis for water billing. In both cases accuracy is vital, so the quality and good maintenance of these devices are very important. Functioning of the water meters is based on three main principles:

1 pressure difference,
2 rotation,
3 magnetic or ultrasonic waves.

Venturi meter
Orifice plate

Flow measurements based on the pressure difference are in fact applications of the Bernoulli Equation for two cross-sections. This principle is used in construction of '*Venturi' meters* (Figure 4.58a) and *Orifice plates* (Figure 4.58b), being mainly for the flow determination in trunk mains.

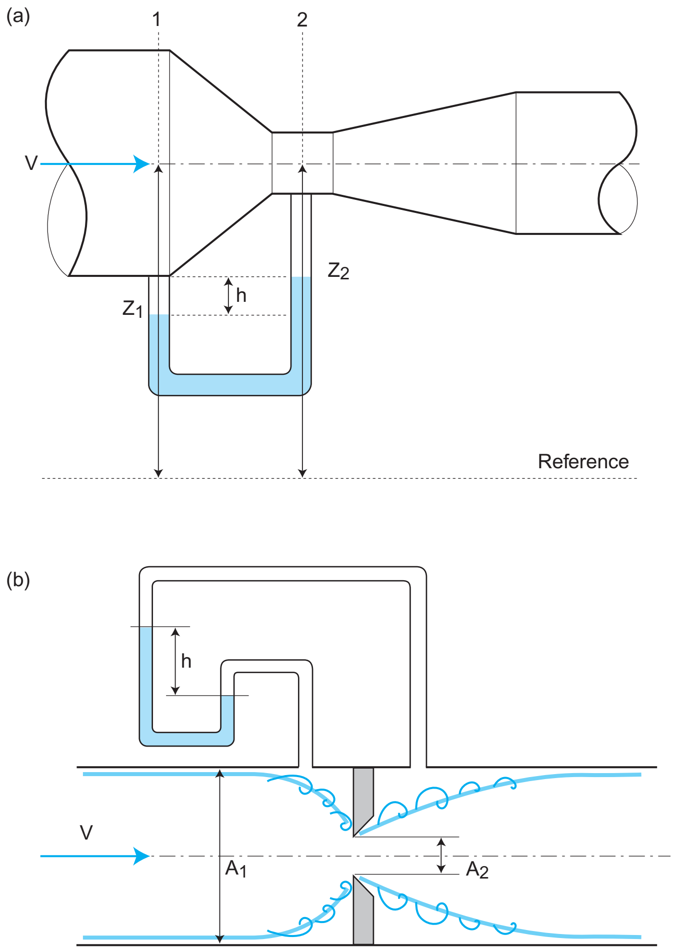


Figure 4.58. Hydraulic flow meters: 'Venturi' meter (a) and Orifice plate (b).

These devices can also be found at major junctions of water transport systems, or within any water distribution system where large quantities are to be measured.

Both types are simple in construction and do not require any electronic equipment; a differential manometer is the only measuring device. Geometry of the cross-sections and the minor loss coefficients based on the pipe contraction/shape of the orifice must be known for determination of the mean velocity (see Appendix 3).

Hydraulic flow meters present a certain obstruction to the flow, generating hydraulic losses and limiting the pipe maintenance. The flow meters that have no moving parts and do not present any physical obstruction are based on either magnetic field or ultrasonic wave measurements.

Magnetic flow meter

Magnetic flow meters (Figure 4.59) create a DC magnetic field that is affected by the water flow. As a result, a small electricity current will be generated proportional to the flow velocity, which defines the velocity profiles within the pipe cross-section. A sensor, that emits pulses to the measuring transmitter, receives the waves indicating the rate of flow.

Magnetic meters are very accurate but rather sophisticated and expensive devices for mass implementation. They are widely used for measurements of bulk flows in pumping stations and on main trunks. Sensitive electronic components may restrict their application under extreme temperatures and wet (humid) conditions, if proper protection is not available.

Ultrasonic flow meter

Ultrasonic flow meters make use of ultrasonic waves to sample the velocity profile within the pipe. The meters used for drinking water are commonly based on the *transit time principle*, which takes the speed of sound propagation in water into account. Two sound transducers are installed along a short pipe distance, exchanging the diagonal sound

Figure 4.59. Magnetic meters (Manufacturer: Krohne).

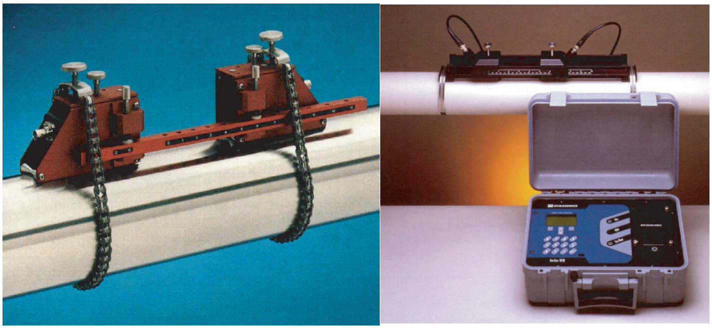


Figure 4.60. Ultrasonic meters (Manufacturers: Controloton – left and Dynasonic – right).

waves in opposite directions. The difference between the sound frequencies of the two signals, which is proportional to the flow rate, will be registered because the sound travelling against the flow will need more time to reach the receiver than the one travelling with the flow. If the transducers are installed on the opposite side of the pipe, the wave exchange will be straight. However, they are more frequently installed on the same side of the pipe (Figure 4.60), creating a refraction of the emitted wave from the pipe wall prior to being received by the opposite transducer, which increases the accuracy of measuring.

Ultrasonic flow meters are less accurate but a cheaper solution than magnetic meters. Their main advantage is the ease of installation that allows mobile measurements on various sections of the network. The pipe diameter, material and wall thickness have to be taken into consideration while calibrating the measuring device.

Inferential meters

Inferential meters are mechanical meters used for flow measurements in small-and medium-size distribution pipes. These meters register the quantity of water passed by rotational speed of a vertical or horizontal rotor or vane, which is then transferred to a counter or register. Larger models are commonly produced in diameters between 40 and 500 mm. Two models are shown in Figure 4.61.

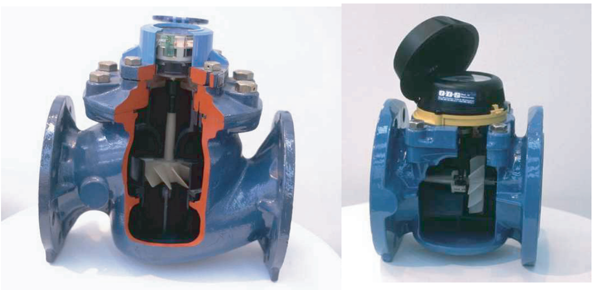


Figure 4.61. Inferential meters with vertical and horizontal axis.

Figure 4.62. Household water meter.

Smaller-size inferential water meters (Figure 4.62) are predominantly used for individual service connections in residential areas. They are produced for pipe diameters of 15–40 mm and can be either the single-jet or multi-jet type.

Volumetric meters

Volumetric meters (also called *positive displacement meters*) are another type of mechanical meters used for billing purposes. The rotation of the moving element placed in a measuring chamber of a known size is converted to the volume of water registered on the display. The moving element can be a circular piston or a nutating disc. The principle for operation of the piston is shown in Figure 4.63.

Under normal operation (maximum working pressures around 100–120 mwc and ambient temperatures 0–40 °C), all small-size mechanical meters are fairly accurate and provide measurements within a ± 2% error margin. This error increases for very low flows, up to a lower limit of the operating range (typically ± 1 l/min); the meter cannot register the flows below this limit. On the other hand, if the flow passing through is too high, the rotating element will be worn out quickly. A model with a proper nominal flow should therefore be selected. The manufacturers usually provide information on the operational flow range, working pressures, accuracy tolerance, etc.

Besides extreme working conditions, an additional concern of mechanical water meters is high water hardness, causing clogging of the rotating elements after some time.

The massive use of water meters in any sizeable distribution system, specifically those used for billing purposes, often demands laborious work in collecting all the records. To save time, different kinds of electronic recorders or loggers can be installed with the flow meters, to enable reading and storage of the measurements (see Figure 4.64). These can be directly connected, or carried to the data processing devices.

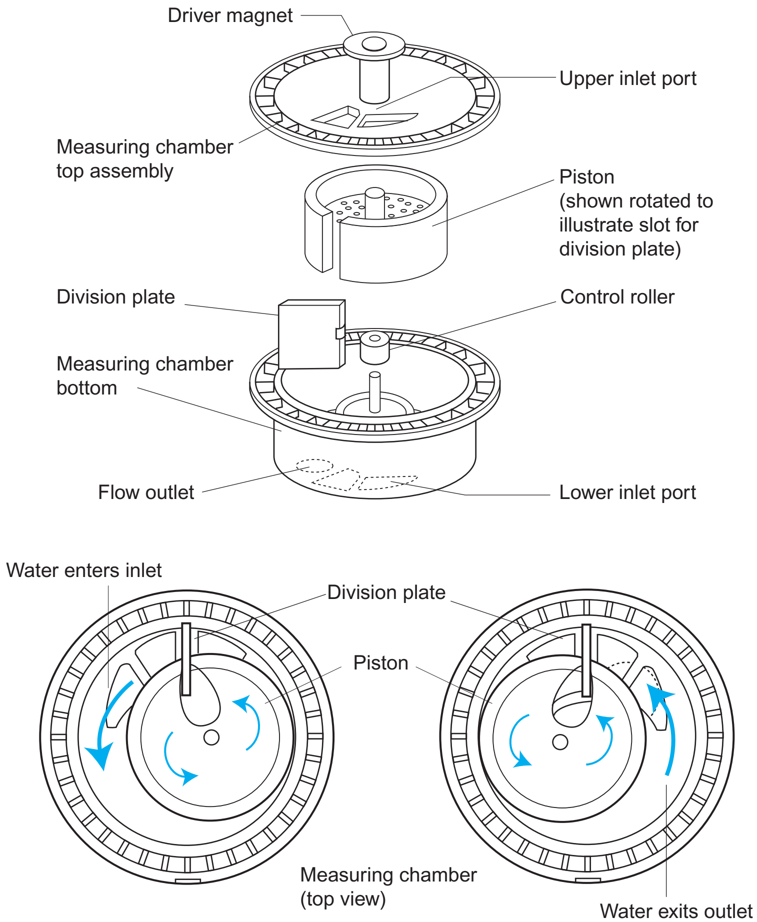


Figure 4.63. Principle for operation of the rotary piston.

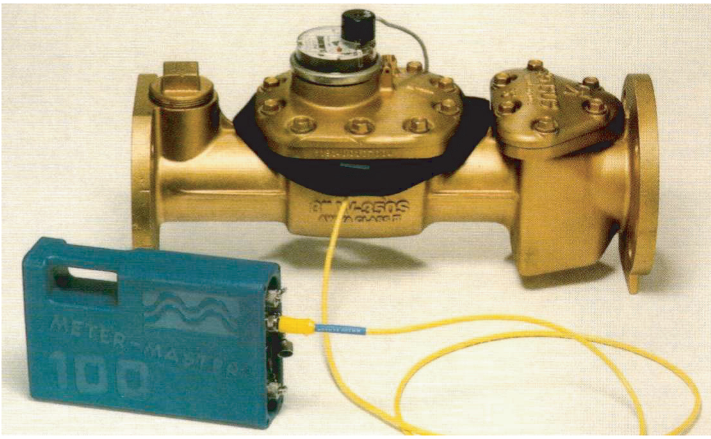


Figure 4.64. Data logger (Manufacturer: Sensus).

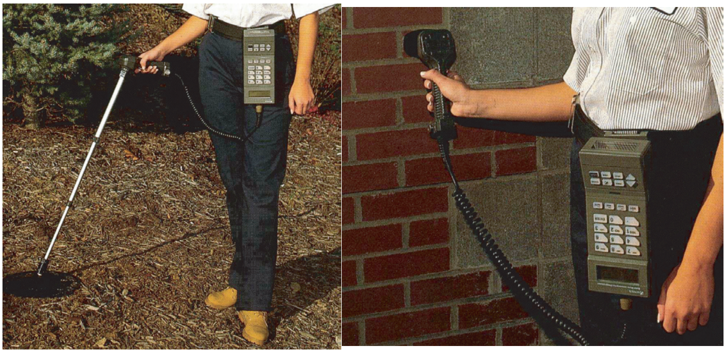


Figure 4.65. Automatic water meter reading (Manufacturer: Sensus).

In recent times, reading on the spot has also been possible without direct access to the water meter, which substantially increases the work efficiency and does not disturb the customers (Figure 4.65). The most recently developed methods are a wireless (radio) connection between the sensor and the logger allowing remote reading (i.e. without leaving the vehicle).

4.5.6 *Fire hydrants*

Fire hydrants are generally distinguished as underground or ground installations (Figure 4.66). The underground installations are better protected from frost and traffic damage, but on the other hand they can be inaccessible when needed, for instance if they are covered by a parked vehicle. The exact position of a hydrant hidden by snow or ice can be detected by placing a plate with coordinates on the wall of the neighbouring house. The plate colour distinguishes between valves and hydrants (Figure 4.67).

Figure 4.66. Ground level- and underground fire hydrants.

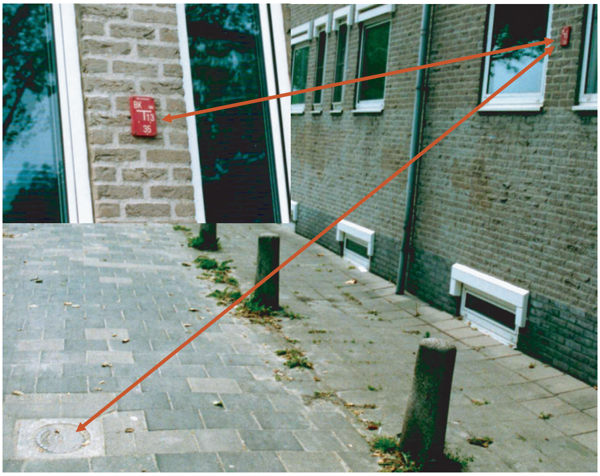


Figure 4.67. Locating underground hydrants (inset shows plate with coordinates).

The hydrants above ground are easy to detect; they are usually painted in bright colours: yellow, red, orange, etc., coding the various capacities of the hydrants. Nevertheless, many people consider them unaesthetic, they can be damaged by cars or vandalised for illegal water use.

This type of hydrant will normally be installed with the main valve, which keeps the barrel of the hydrant dry if not in operation. A small drain at the bottom allows emptying of the barrel after the hydrant has been used. The advantages of such a set-up are that a potentially damaged hydrant is not going to leak. Moreover, illegal water use is impossible without access to the (underground) valve and finally, the freezing of water in the hydrant is prevented. Alternatively, the wet barrel hydrant will be full with water all the time, allowing potential risks but also more prompt operation.

Hydrants are usually located at intersections between streets in order to provide easy access from various directions. In a street, the distance between the two closest hydrants is around 100–200 m. Hydrants should not be placed too close to the buildings as the vicinity of fire or risk of collapsing buildings might prevent their use. To avoid damage from traffic, they should also not be located too close to the road.

The required capacity and pressure for hydrants vary from case to case and these are related to potential risks and consequences from fire. Generally, fire requirements are within 30–50 m^3/h, occasionally up to 100 m^3/h, assuming minimum working pressures above 10–15 mwc. The pressure criterion is usually less of a concern, as the fire fighting engine will normally be equipped with a booster pump. It is however logical to expect that the pressure in the distribution system will drop temporarily as a result of fire fighting, affecting the neighbouring consumers to some extent. In the worst cases a vacuum can be created in the system, causing

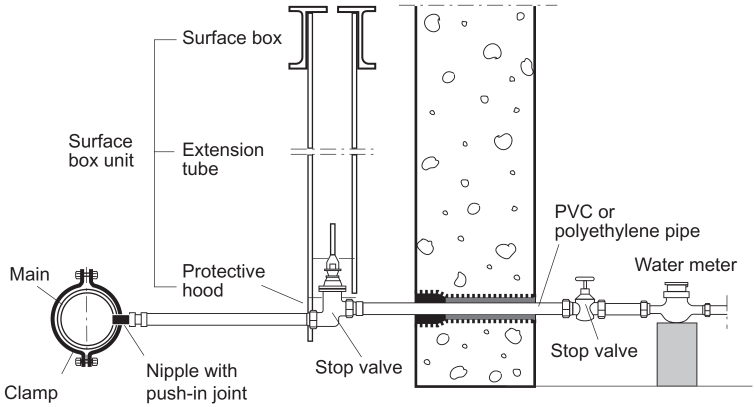


Figure 4.68. House connection with stop valve.

backflow contamination or surge pressures. This situation can be prevented if the hydrant is connected to a fire fighting engine that has a (balancing) tank from where the water is drawn to extinguish the fire. Finally, the hydrants should be closed slowly in order to prevent surge pressures.

Apart from irregular situations, hydrants may be used for other purposes such as cleaning of pipes, leakage control, flushing of streets, etc. Pipe networks with hydrants can also be separated from the drinking water distribution network.

4.5.7 *Service connections*

Service connections link users with the distribution system. The standard set-up consists of: connection, service pipe, outdoor – and indoor-stop valve and water meter (Figure 4.68). In newer installations, a non-return valve may be added as well.

A set-up with a stop valve used for flexible pipes is shown in Figure 4.68. Connection to the distribution pipe can be carried out on the top of the pipe, from the side, with or without saddle, etc. Frequently used types of saddles for DI and PE pipes in The Netherlands are shown in Figure 4.69. Usual diameters of the service pipes are 15–50 mm. Part

Figure 4.69. Types of saddles used for service connections in The Netherlands.

Figure 4.70. Service connection made with PE on a PVC pipe under pressure.

of the pipe passing through the wall of the building will be sleeved by elastic and watertight material (plastic, impregnated rope, bitumen, etc).

Service connections are commonly installed on distribution pipes in operation (i.e. under pressure). An example of the method of vertical connection applied in The Netherlands is presented in Figure 4.70.

4.5.8 *Indoor installations*

Indoor systems comprise all the piping, tap points and appliances within dwellings. In higher buildings, a central pressure boosting system is a compulsory requirement. The task of this installation is to bring pressure to the upper floors of the building. A tank can be constructed on the top of the building, in order to maintain the pressure but also as a reserve for fire protection. The example from Figure 4.71 shows a solution by which the pressure from the distribution pipe supplies only the lower floors, while the middle and the upper floors are supplied from the reservoir on the top. Two pumps are installed, one to fill the reservoir and the other to boost the pressure on the highest floors. Regarding house installations, there is a whole variety of configurations related to the habits of water use. An example of a European set-up is shown in Figure 4.72.

4.5.9 *Engineering design of storage and pumping stations*

The engineering design of storage and pumping stations involves experts specialised in construction, mechanical and electrical equipment. Besides sufficient amounts of water, a well-designed storage should:
– maintain non degraded water quality,
– provide reliable operation,
– allow easy maintenance.

To reach the above goals, the following engineering aspects of storage constructions have to be considered (Williams and Culp, 1986):
– Water circulation through reservoirs should be provided by baffles, or by placing inlets and outlets on opposite sides of the reservoirs, with outlets near the bottom.
– Reservoirs should allow maintenance without loss of pressure in the distribution system. If there is a single reservoir, it should be divided into compartments so that at least one part of the volume is available for use.
– Proper protection of contamination should be provided. All finished water storage structures should have suitable water tight roofs or covers to exclude surface waters, birds, animals, insects and excessive dust. However, sufficient ventilation should be enabled but with vents protected from the same impacts. The drains should discharge to the ground surface with no direct connection to a sewer or storm drain.

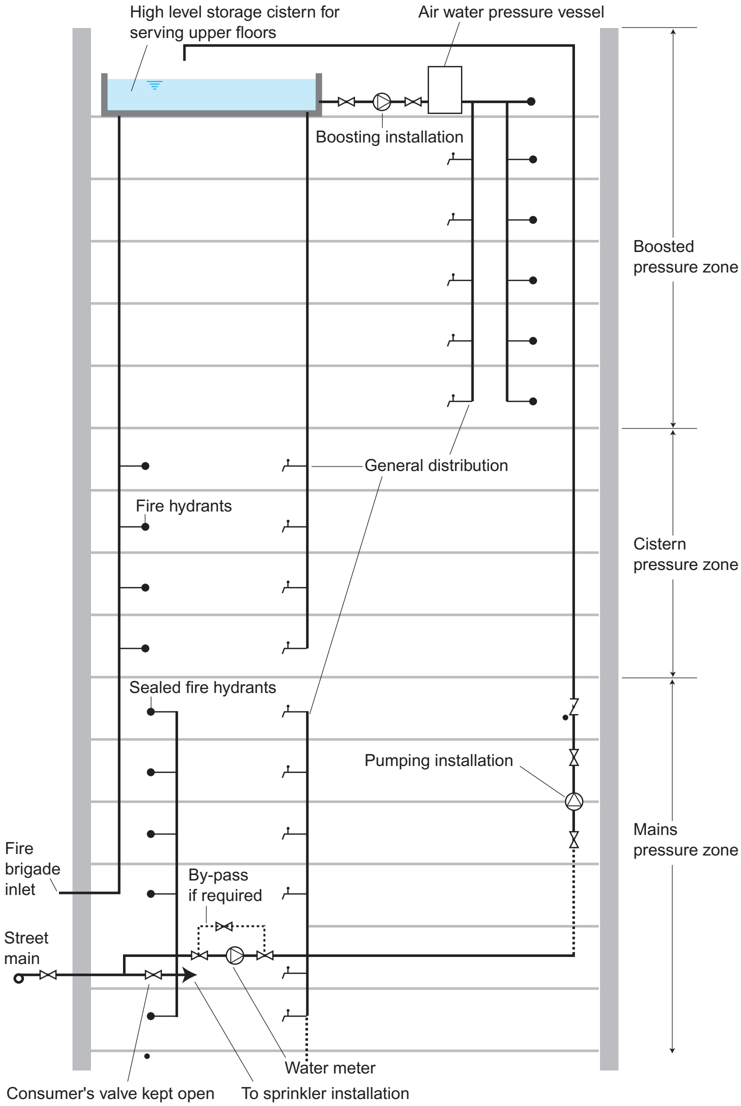


Figure 4.71. High building with roof tank.

– Water in the tank should be isolated from the impact of excessive temperatures.
– Pipes connected to the tank should be equipped by valves installed outside the storage (except the overflow). Any pipe running through the roof or sidewall should be welded or properly gasketted in metal tanks or connected to built-in concrete castings with flanges. Corrosion protection should be introduced wherever necessary. Entry of silt through inlet pipes should be prevented.

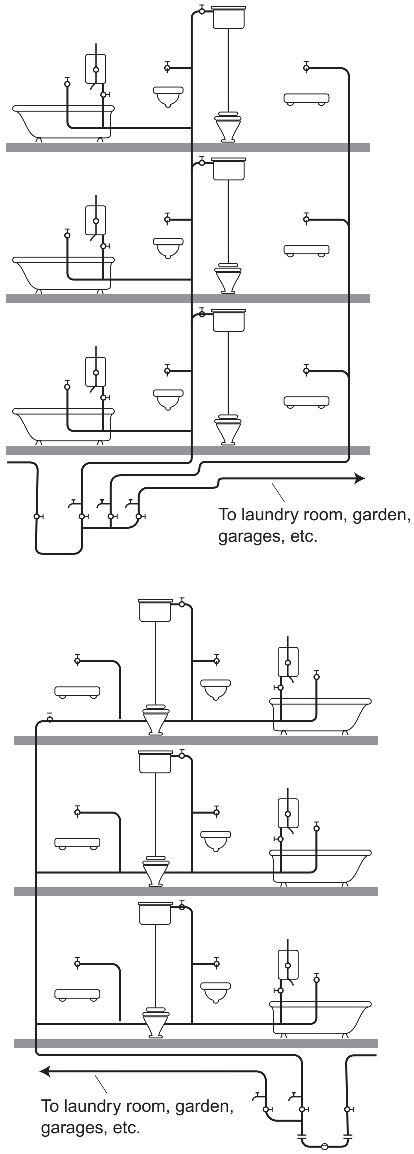


Figure 4.72. Indoor installations.

– Access to the storage interior should be convenient for cleaning, maintenance and sampling. On the other hand, fencing, lock on access manholes and other necessary precautions should hinder vandalism and sabotage.
– Adequate measuring equipment should monitor water levels in the tank.

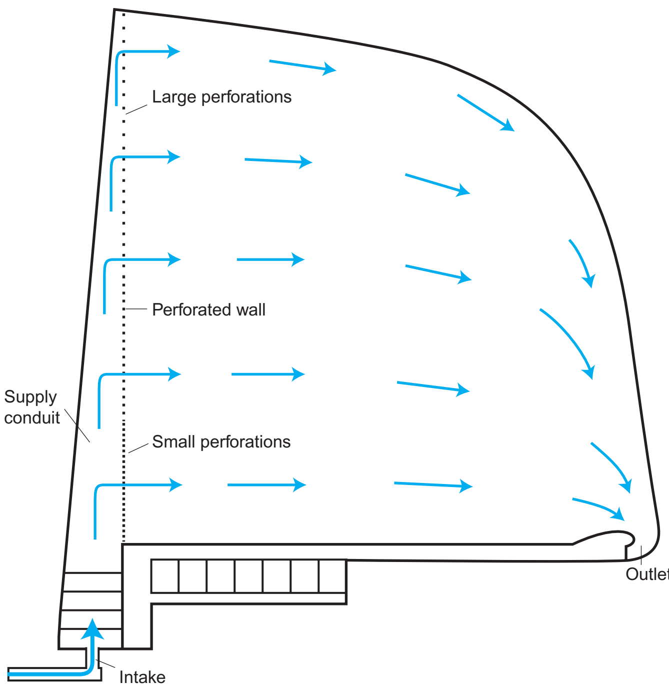


Figure 4.73. Water reservoir in Munich (Germany).

The cross-section of storage reservoirs is usually of rectangular or circular shape. An interesting construction was carried out in the case of the underground storage reservoirs in Munich (Germany), shown in Figure 4.73. The inlet structure of this tank is a perforated wall with openings of various diameters that have to create unequal flow distribution. The shape of the tank, causing various retention times and mixing of the water, further amplifies this. More traditional inlet and outlet arrangements are shown in Figures 4.74 and 4.75 (Obradović, 1992). In cases where the water is pumped from the reservoir, the suction pipe will be positioned below the bottom level of the tank in order to minimise its 'dead' volume and at the same time to guarantee the minimum suction level for the pumps (Figure 4.75/left).

Regarding pumping stations, the main engineering aspects are (Williams and Culp, 1986):
– accessibility and layout of the station,
– foundations and vibration,
– drainage,
– acoustic insulation,
– heating, ventilation and lighting,
– corrosion,
– protection against vandalism,
– health and safety.

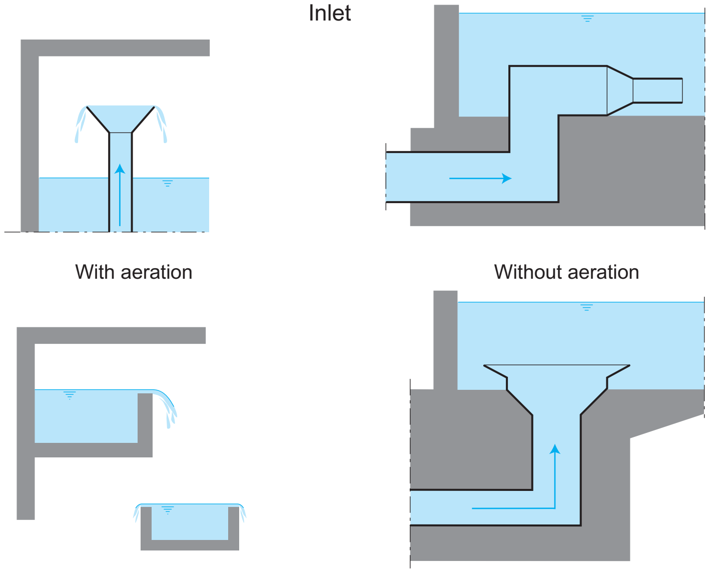


Figure 4.74. Reservoir inlet arrangements.

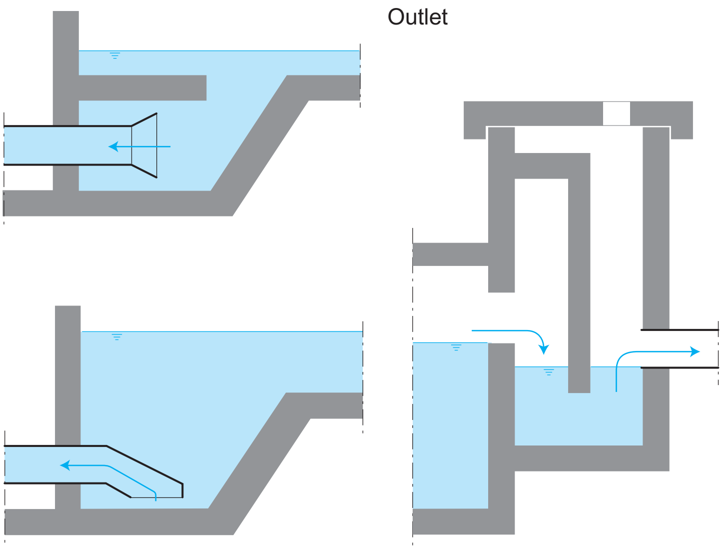


Figure 4.75. Reservoir outlet arrangements.

The facilities for installation, servicing, dismantling and removal of pumps should be planned while keeping the size of the whole station at a minimum. The design of the pipe work should be flexible to accommodate tolerances in assembly, and also to maintain the units without stopping the operation of the whole station; possible future extensions should also be born in mind (Figure 4.76).

Figure 4.76. Piping in pumping station.

Figure 4.77. Horizontal pumps with rubber rings to dampen vibrations.

Well-constructed foundations are essential for good operation, particularly where the pump and motor are not integral. Where vibration is expected, foundation blocks should be isolated with damping material from the rest of the structure. Isolation of the piping from vibrations should also be provided (Figure 4.77).

Most of the design requirements can successfully be tackled if the station is constructed partly or wholly in the ground. Savings in architectural finishes are likely to offset the additional costs of burying the structure, providing good hydraulic performance (NPSH), sound, heat and frost insulation, protection against vandalism, etc. However, the disadvantages are that access to the station is more difficult, the drainage system in the station needs additional pumping, condensation may be a hazard for electrical equipment, etc.

Developments in remote control and telemetry have virtually removed the need for stations to be permanently manned; the buildings for pumping stations are required primarily to accommodate and protect the machinery, providing its smooth operation and maintenance (Figure 4.78).

Figure 4.78. Unmanned booster pumping station.

4.5.10 *Standardisation and quality assessment*

The need for standardisation is clear for water distribution companies:
- Regarding planning and design, it allows engineers to be more precise in drafting specifications. The required testing and inspection of materials guarantees quality of applied materials.
- Regarding O & M, it reduces difficulties caused by the diversity of the installed material.

Hence, the first step towards standardisation is the elimination of variety within certain products. Thereafter, an evaluation of the technical characteristics of the component follows, namely the fitness for the purpose that must result in a sufficient and guaranteed quality level for the use of products.

The quality assessment, which is implicitly the result of standardisation, is very important in this phase. In any manufacturing process there will be a deviation within certain limits and the products have to be tested; variations in quality may not be acceptable.

The level and frequency of testing depend on several factors but predominantly on the degree of certainty with regard to the consequences caused by inadequate quality. There are basically three different principles (van der Zwan and Blokland, 1989):
- batch testing,
- testing and assessment by the manufacturer's internal control, followed by surveillance,
- internal assessment by the manufacturer.

By batch testing, a product sample is tested and a verdict on its compatibility with the specifications is issued.

The second principle is a sample testing of the product according to a prescribed method approved by the manufacturer's quality control. This testing is regularly followed by surveillance in the form of quality control in the factory, but also by audit testing from both the factory and open market.

The third system concerns the manufacturer's capability to produce consistently in accordance with required specifications. The manufacturing methods, facilities and quality control are assessed and approved in respect of a discrete technology that is capable of delivering products of constant quality.

It has to be realised that any quality control requires qualified personnel, and accurate measuring and testing equipment. The factory justifies the cost of such investment only if it increases productivity in the long term. On the other hand, the processes that can provide an initially low level of rejected products usually include modern technologies that are also expensive. Hence, the manufacturer is essentially interested in introducing as much quality control as necessary to achieve good sales figures. Moreover, the absence of real market competition or/and supervision may lead to the relations between manufacturers and water supply companies becoming based exclusively on mutual trust and willingness for cooperation. Two possibilities to prevent this are:
– the centralisation of testing,
– a certification procedure.

Centralisation of testing can be obtained by establishing an institution that will assist the water sector with services related to quality control. This offers obvious advantages:
– lower expenses for quality control,
– availability of proper equipment at the required time,
– continuity of testing and inspection,
– development of experience in the field,
– the exchange of information.

Certification of industrial performance is not only important for the water industry as a client, but it is also an assurance to the manufacturer that the required quality has been agreed and is acceptable for everyone. The institution responsible for centralised testing is also usually the one that issues certificates. It is the ultimate controller of the production and responsible for the quality in general.

The experience of centralised control and certification has shown very positive results in The Netherlands, where these activities have been carried out by the KIWA institute since 1948.

Nowadays, the vast majority of products for the water industry carry the KIWA-label, which is almost a prerequisite for their implementation in any distribution network in the country. Accredited by the Dutch Council for Certification, KIWA issues a large number of quality certificates not only for waterwork products but also for building components, process and quality systems, etc. The certificate can be issued for:
– the quality of the product,
– an assessment of the manufacturing process,

– an assessment of internal quality control scheme,
– an assessment of toxicological aspects of the production.

The certification procedure follows standard steps in each case, shown in Figure 4.79.

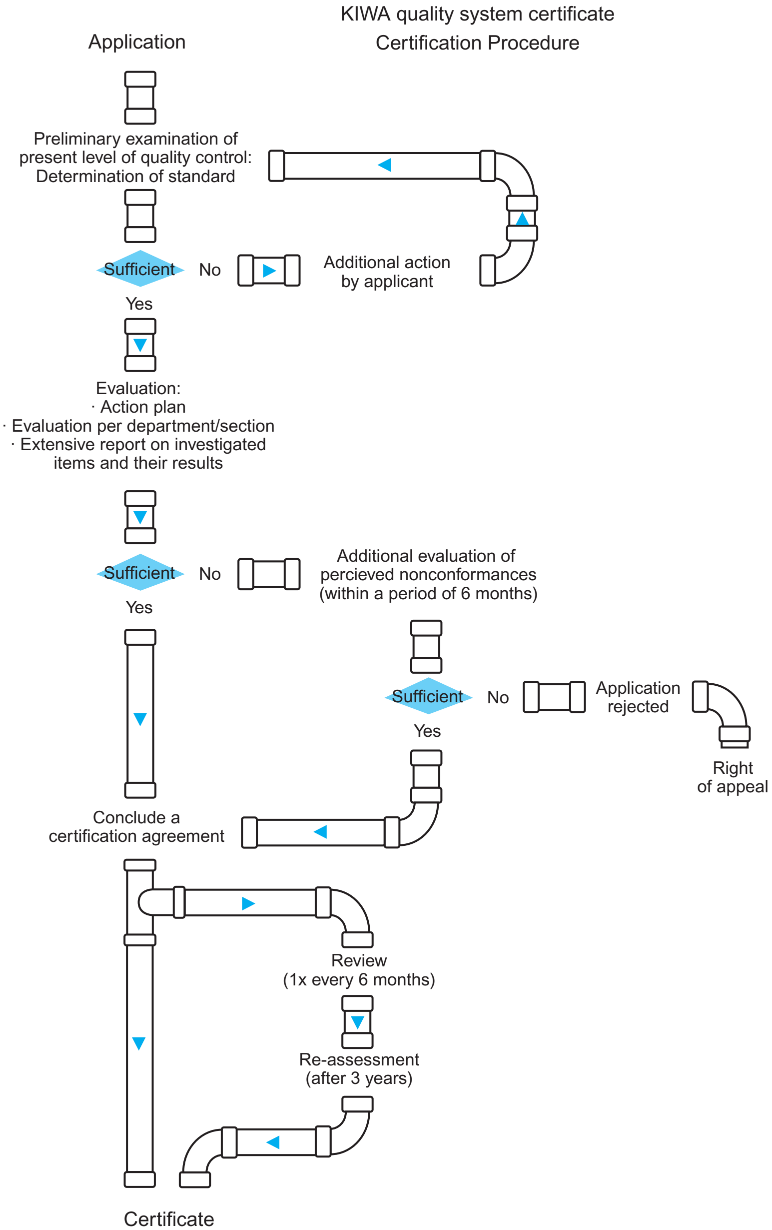


Figure 4.79. Certification procedure (Source: KIWA).

CHAPTER 5

Network Construction

Network construction comprises the following steps:
1 site preparation,
2 excavation,
3 trench dewatering,
4 pipe laying,
5 jointing,
6 backfilling,
7 testing & disinfection.

After the site has been prepared, all the other steps are conducted simultaneously at various sections of the pipe route; at its end, the pipes are tested; a few pipes further, the backfilling takes place; and at the same time at the preceding section the pipes are jointed, etc. This coordinated method of working is important in order to shorten the total duration of the construction, reducing both the cost and disturbance. The laying of a few sections of steel pipe is shown in Figure 5.1.

Pipes can also be laid above ground or in tunnels, which then require adapted laying techniques such as the use of casings, anchorages and supports, etc. Some typical principles and solutions are briefly presented in this chapter.

Figure 5.1. Laying of steel pipe.

5.1 SITE PREPARATION

Pipes can be laid only when the route is completely clear. Site preparation in urban areas can be a complex task where cooperation with other utilities is very important. Works on water, electricity, gas, road or other infrastructure are often carried out simultaneously.

Before the work can commence, mutual agreement should be obtained about the working area so that other daily activities are not significantly affected during the construction. Proper signalling, footpaths and crossings for pedestrians, signs and warnings, a restricted access to the equipment in operation etc. must be provided during the entire period of work.

Pipes will be tested prior to leaving the factory and should also be tested after reaching the site in order to check for possible damage resulting from transportation. Further damage to the pipe is possible during the process of unloading, stacking and/or stringing along the laying route. The dropping of pipes, pipes striking each other, bundling pipes too high and stacking them on an uneven surface or without proper support will all have a negative effect. Each scratch on the external or internal coating of a metal pipe is a potential source of corrosion. Cement-based pipes are very vulnerable to impact damage and plastic pipes, although lighter, are not an exception in this respect; scratches on PVC reduce the pipe strength. Hence, a final check is necessary for each pipe before it is put into position.

Pipes and fittings waiting to be installed should be kept clean in a fenced storage as a protection against potential theft and vandalism (Figure 5.2).

Before excavating paved surfaces and roads, the cutting of edges of the trench has to be done to avoid damage to surrounding areas. If traffic

Figure 5.2. Pipe storage on the construction site.

loads allow, the pipe route will be located alongside the road, preferably not too far from it, which reduces damage to the pavement resulting from excavation. Breaking the surface is usually carried out by pneumatic hammers. Large pieces of concrete and asphalt will be removed from the site as they will not be used for backfilling. If the surface is not paved, the topsoil is usually removed by scrapers and stacked for use in the final reinstatement of the site.

5.1.1 *Excavation*

Excavation is the most expensive part of pipe laying. The choices of technique and trench dimensions are therefore very important factors that will affect the total cost. The preferred excavation method depends on
– available space on the site,
– soil conditions,
– width and depth of the trench.

Excavation is commonly carried out by mechanical excavators (Figure 5.3). In areas where there are obstructions (e.g. other services are in the trench) or access for the machine is restricted (small streets, busy traffic, etc), excavation by hand might be required (Figure 5.4). For smaller trenches (up to 300 mm wide and 1 m deep) vacuum excavation can be used. After breaking the surface and removing the top layer in the conventional manner, a special pneumatic digging tool is used. With this method, the soil is then removed through a flexible hose.

Care has to be taken during the work:
– to stabilise the walls, either by battering or shoring,
– to clear the trench edges of chunks of rock or earth that could potentially damage the pipe or hurt the workers,
– to leave enough space between the trench and pile of excavated material,
– to keep the work as dry as possible.

Figure 5.3. Mechanical excavation in sand.

Figure 5.4. Manually excavated trench.

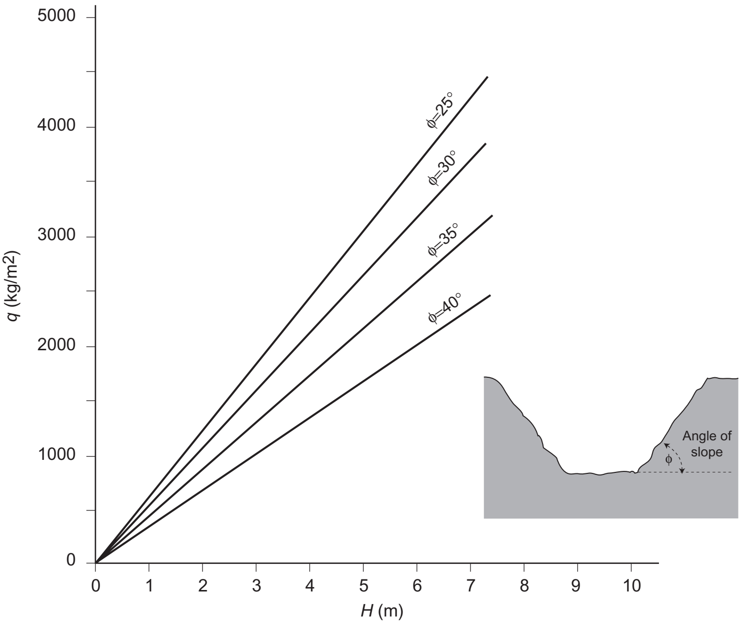


Figure 5.5. Trench slopes (Pont-a-Mousson, 1992).

Batter-sided trenches are rarely used in urban areas because of the space needed. Where possible, the angle of slope should depend on the trench depth and soil characteristics, as shown in Figure 5.5.

Different techniques of shoring can be applied by (Brandon, 1984):

1 prefabricated wooden panels (jointed or single),
2 wooden or metal sheets,
3 pile driven sheets.

The choice of technique, dependant on the soil conditions, is often prescribed by laying regulations. Three groups of soils can be distinguished regarding their suitability for excavation (see Figure 5.6).

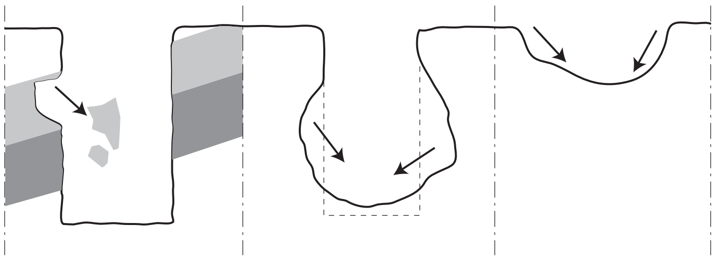


Figure 5.6. Soil types.

Rocks

Rocks are extremely cohesive materials but the possibility of collapse cannot be excluded. Cracks are sometimes present, which can result in rocks falling. Excavation is difficult in this type of soil.

Friable soils

Friable soils are the most common soils. A certain degree of cohesion allows them to hold together for a while during excavation. However, these soils are very sensitive to water, and collapse of the trench walls caused by the vibration of the equipment is also possible.

Non-cohesive soils

Non-cohesive soils are soils without any cohesion (e.g. dry sand, mud or freshly restored backfill), which collapse almost instantly. Protection against the danger of collapse is therefore essential.

Shielding

The *shielding* technique can be used in rocky and friable soils, in the absence of shoring. By this method, the laying and jointing work takes place in a partly open steel box that is pulled throughout the trench as the work progresses. The sidewalls of the box do not prevent occasional caving in of the soil, as the width of the box is smaller than the trench width in order to be able to pull it smoothly. The main objective here with this method is the protection of the workers.

How much trench is excavated depends on the time necessary for pipe laying and backfilling. Normally, the trenching is excavated a day or two ahead of the pipe laying, depending on the laying methods applied. However this should not be carried out too far in advance, as empty trenches may accumulate rainwater and are potentially dangerous, especially outside working hours.

The width of the trench at the bottom depends on the pipe diameter. An additional space of 0.3–0.6 m around the pipe (external diameter) should be provided for shoring and jointing works.

Extreme temperatures can have an impact on the operation of water distribution systems, not only by affecting the water consumption but also by causing pipe damage either by freezing or very high temperatures. While deciding on the optimal trench depth, care should be taken to minimise the temperature impact on pipes and joints. On the other

Table 5.1 Soil cover over pipes.

Country	Depth (m)
Austria	1.0–1.5
Belgium	0.8–1.0
Finland	2.1–2.5
Germany	1.1–1.8
The Netherlands	0.8–1.0
Switzerland	1.2–1.5

hand, increasing the depth beyond what is really essential is more costly, not only during installation but also in the maintenance phase. Some degree of pipe burst under extreme weather conditions is always acceptable if the repair can be conducted quickly and without disturbance to a large number of consumers.

In general, the minimum cover over the pipe crown in moderate climates are
– 1.0 m for transmission lines,
– 0.8 m for distribution pipes,
– 0.6 m for service pipes.

For frost prevention, pipes are laid deeper in areas with a cold climate, sometimes up to 2.5–3 m, which depends on the degree of frost penetration in the ground. Alternatively, pipes in shallow trenches can be laid with thermal insulation. In extremely hot climates, the pipes will also be buried deeper, mainly to preserve the water temperature. Examples from practice are shown in Table 5.1.

The excavated material is deposited alongside the trench if it is going to be used for backfilling. Its location should not be too far from the trench but also not too close, as it exerts pressure on the trench wall, risking its collapse. Moreover, it also limits the movement of the workers. In general, approximately 0.5 m space should be left free for deposited material.

Tunnelling

Excavation for laying pipes passing under roads, railways and watercourses is done by *tunnelling*. The special reason for this is to protect the surrounding area from erosion caused by the pipe burst or leakage, which can have catastrophic consequences. Second, the pipe is protected in this way from soil subsidence and vibrations caused by traffic, and maintenance can be carried out without interruptions or breaking of the surface.

Excavation of tunnels is a very expensive activity. In this situation thrust boring is applied, whereby a rotating auger moving the excavated material backward pushes a steel shield pipe forward. New lengths of pipes are welded or jointed together as the tunnelling proceeds, finally appearing at the other side of the crossing.

Cut and cover method

The thrust boring technique is successful for short lengths of tunnels, up to 100 m, and for pipes of maximum 2500 mm diameter (Brandon, 1984).

For longer lengths and larger diameters, a tunnel should be constructed by traditional methods. These structures can also serve to accommodate several pipes, usually water mains carrying large quantities of water. In rock, the tunnel section can be a vertical wall lined with concrete; for other soils circular sections formed by reinforced concrete segments are common. When the tunnel is shallow it can be constructed by the *cut and cover method* and in this situation a reinforced concrete box culvert is a more suitable solution.

5.1.2 *Trench dewatering*

The normal method of removing water as it enters the excavation is by pumping (Figure 5.7). Sand and silt in unstable soils are mixed with water and carried out as well. If this continues over a period of time, there is a danger of subsidence in adjacent ground. In such situations, the removal of ground water can be carried out by using well point dewatering equipment (Figure 5.8). The water is collected through perforated suction pipes put in the ground below the lowest excavation level. All suction pipes are connected to the header pipe, which transports the water by vacuum created by a well point pump. The equipment used for this method is shown in Figure 5.9.

Figure 5.7. Trench dewatering by pumping.

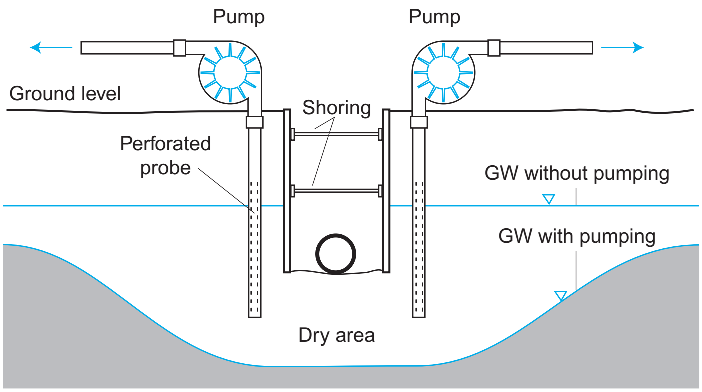


Figure 5.8. Principle of the well point method.

Figure 5.9. Application and equipment for the well point method.

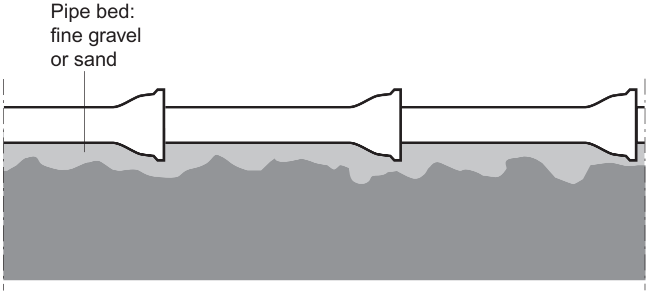


Figure 5.10. Pipe bed.

Although proven to be very efficient in the case of non-cohesive soils, the well point dewatering method can rarely be used in impervious soils because the water is not able to flow to the extraction points. Electro-osmosis, forcing the water by means of a passage of electrical current to a dewatering point, may be successful in maintaining vertical sides in wet unstable silt.

5.2 PIPE LAYING

5.2.1 *Laying in trenches*

The trench bottom provides the pipe's foundations. In homogeneous, even and well-consolidated soils, pipes can be laid directly on the bottom. The pipe should touch the ground surface with its entire length. To facilitate this, the space around joints should also be excavated. In rocky soils, a pipe bed of 15–20 cm should be provided (Figure 5.10). Depending on the pipe material, the bed can be made of sand, gravel or dry concrete, which assumes that the surface of the trench bottom is even and well compacted

When it is necessary to lay on less stable ground, pipes should be supported on piles based on a stable material, if such materials is to be found at a depth less than 1.5 m. Care should be taken to avoid point loads being transmitted to the pipes (particularly in the case of PVC pipes).

Piles can also provide support to the pipes in waterlogged grounds. If this is not sufficient, lowering the ground water table can be achieved by laying a drain alongside the trench at a depth of 0.5 m below the pipe invert. The pipe is bedded on the reinforced concrete raft placed across the trench bottom, which ensures its stability. An example of concrete transportation pipes laid on wooden piles is shown in Figure 5.11.

Most pipes are still laid individually in the trench. With the increased use of flexible pipes, the technique of laying large sections of distribution mains is becoming more common. The placing of pipes on the prepared bed in a position ready for jointing requires appropriate equipment and skill (Figures 5.12 and 5.13). The precise laying procedure depends on the

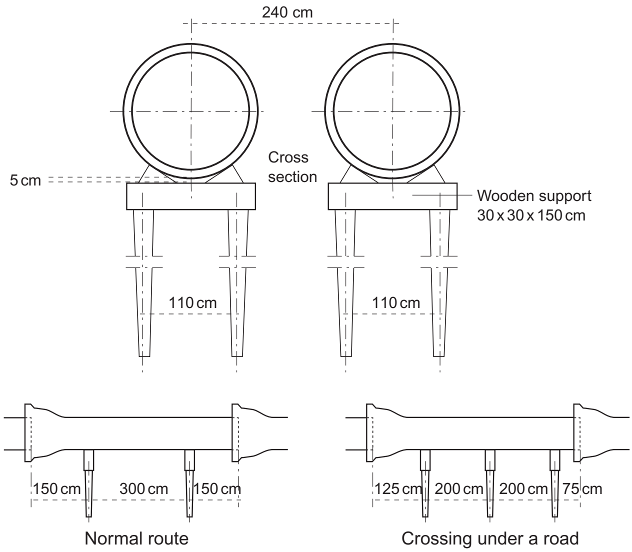


Figure 5.11. Pipes laid on wooden piles.

Figure 5.12. Testing of external coating.

pipe material; the advice of pipe manufacturers must be taken into account here. The entering of ground- or rainwater into the pipeline is highly undesirable, so pipe stoppers should be used if the work has to be halted, for example, at the end of the day. In highly corrosive ground, metal pipes (and joints) can be sleeved into a polyethylene film at the time of laying as an additional protection to the external coating, as shown in Figure 5.14.

5.2.2 *Casings*

Different principles of casings are possible; two methods are shown in Figures 5.15 and 5.16.

Old pipes can sometimes be used as casings for the new pipes (Figure 5.17). This solution will probably reduce the maximum capacity of the line, although the smaller diameter is partly compensated for by the decreased roughness values of the new pipe. Special care should be paid to the jointing of the new pipes in order to make the route leakage free, as there is little space for any possible future repairs or maintenance.

5.2.3 *Laying above ground*

The following aspects should be considered when laying pipes above ground:
1 the design of the support system,
2 the accommodation of thermal expansion,
3 the anchorage of components subjected to hydraulic thrust,
4 protection against freezing (where necessary).

Figure 5.13. Pipe positioning in a trench.

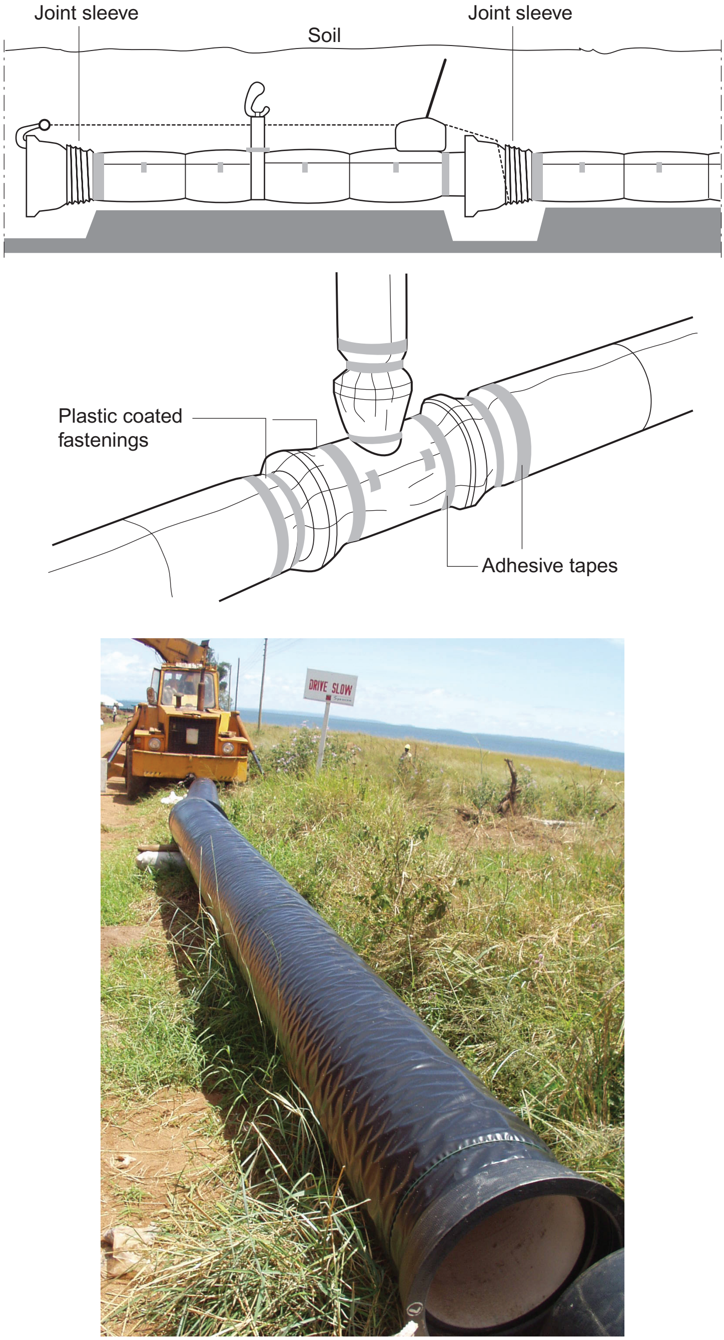


Figure 5.14. Protection of pipes and joints (Pont-a-Mousson, 1992).

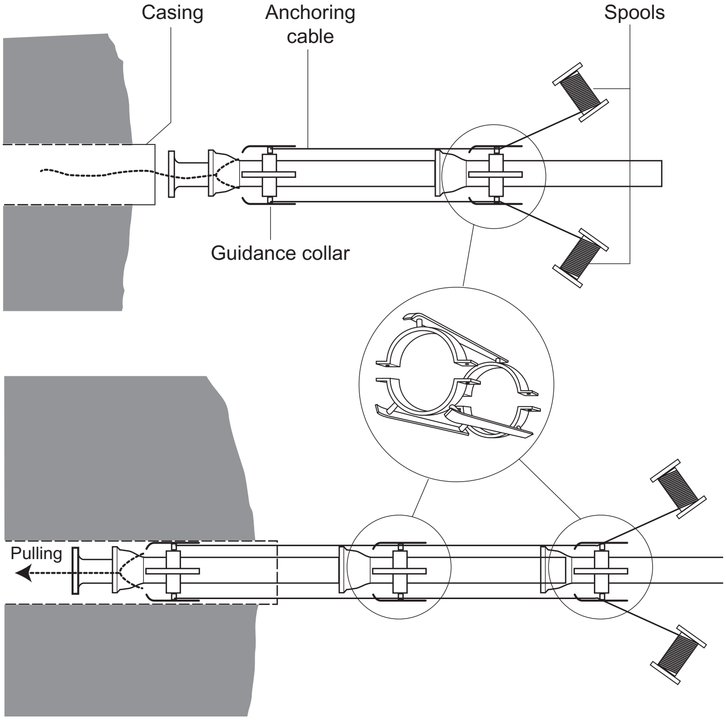


Figure 5.15. Pipe casing (Pont-a-Mousson, 1992).

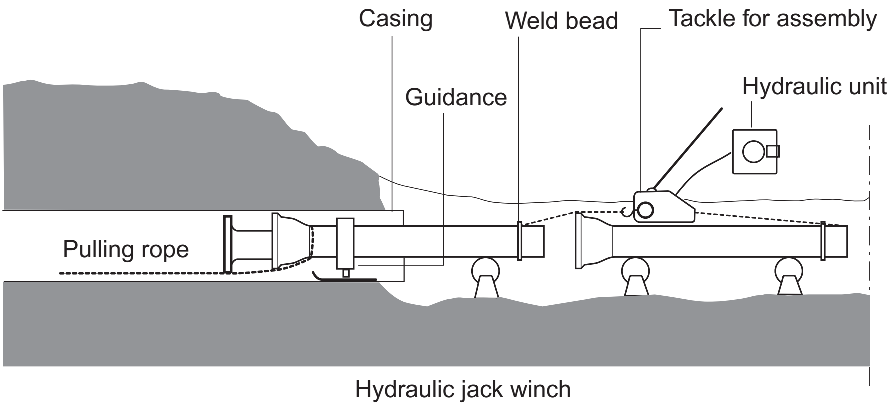


Figure 5.16. Pipe casing (Pont-a-Mousson, 1992).

Some examples of the laying of DI pipes in tunnels and crossings are shown in Figures 5.18–5.20.

Figure 5.17. Casing of a PE pipe in an old CI pipe.

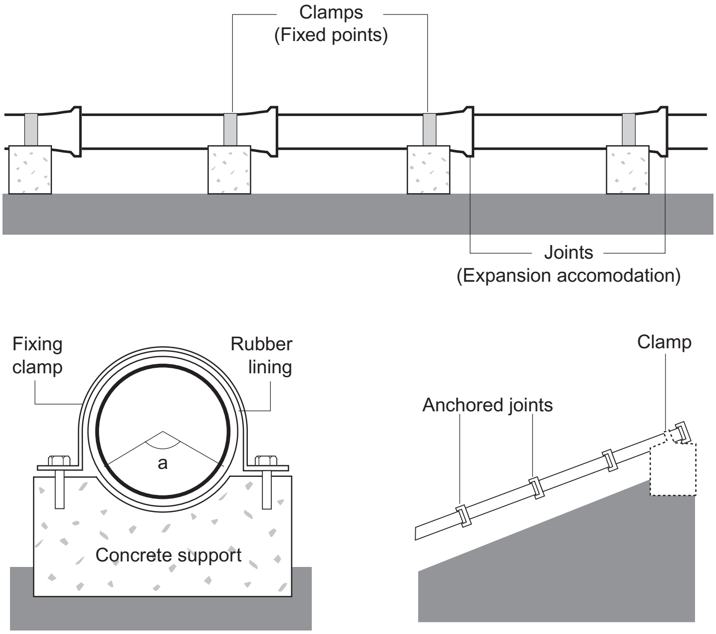


Figure 5.18. Pipe laying on a concrete support (Pont-a-Mousson, 1992).

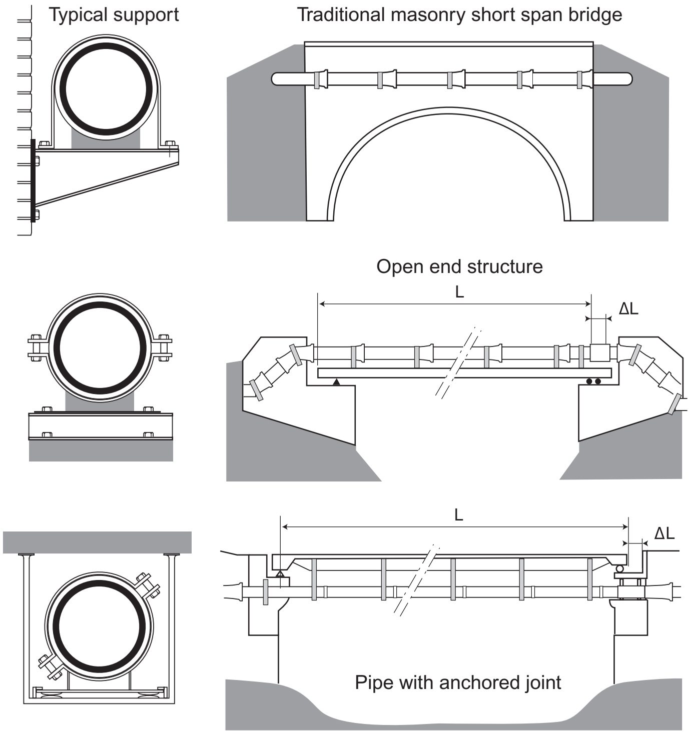


Figure 5.19. Pipe laying at a crossing (Pont-a-Mousson, 1992).

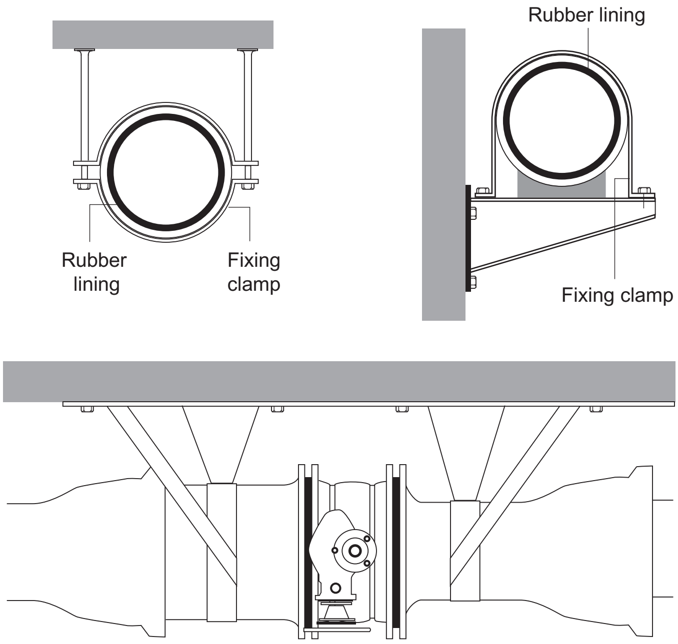


Figure 5.20. Pipe laying when crossing a road (Pont-a-Mousson, 1992).

5.3 PIPE JOINTING

Examples of jointing principles and tools are shown in Figures 5.21–5.24.

5.3.1 *Flanged joints*

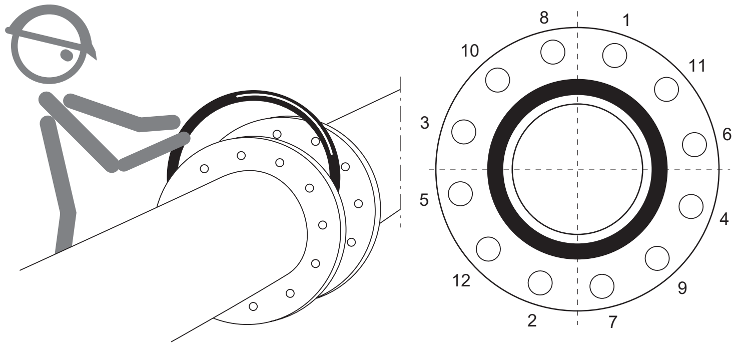


Figure 5.21. Pipe jointing using flanged joints.

5.3.2 *Gland joints*

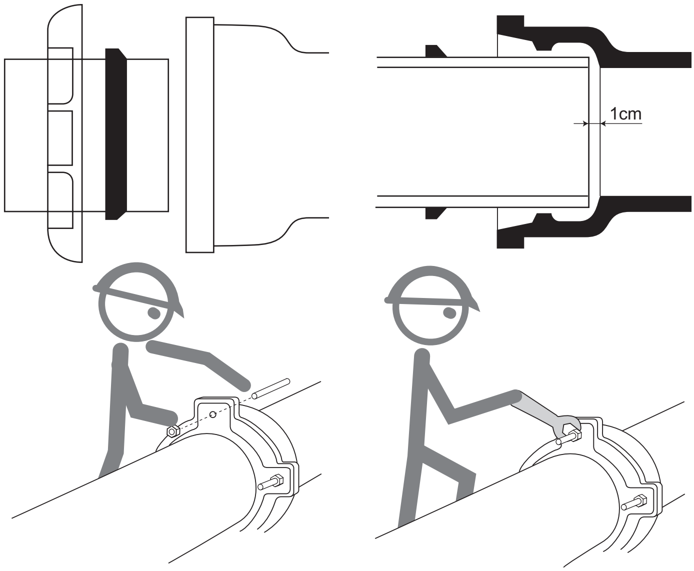


Figure 5.22. Pipe jointing using gland joints.

5.3.3 *'Push-in' joints*

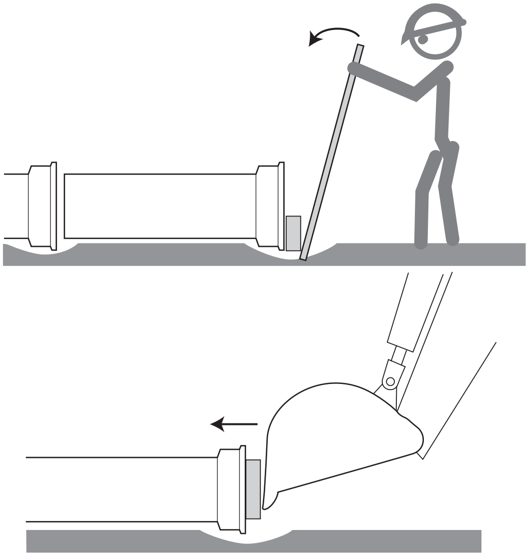


Figure 5.23. Pipe jointing using 'push-in' joints.

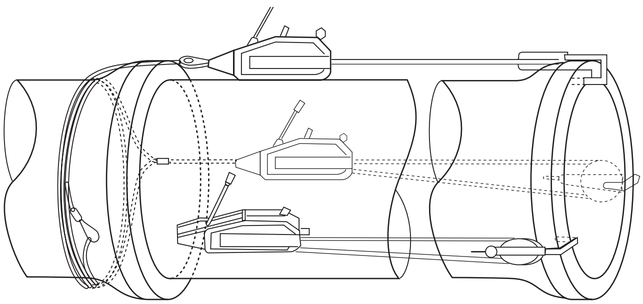


Figure 5.24. Jointing equipment (Pont-a-Mousson, 1992).

5.3.4 *Anchorages and supports*

After the pipes have been laid and connected, the concrete anchorage and support structures must be cast before backfilling is completed. Anchor blocks are designed depending on the pipe configuration and soil characteristics. In principle, each case is considered separately (Figure 5.25). The design takes into account the forces involved and the result is usually expressed as a volume of concrete required to carry the thrust. The water pressure taken into consideration for this calculation is the maximum anticipated one, with an additional safety factor in case pressure surges are expected.

Concrete should be placed and consolidated against undisturbed soil and around the pipe or fitting to achieve a good bond. Care must be taken when filling with concrete to keep joints clean. The position of the thrust blocks for some typical bends and junctions is shown in Figure 5.26.

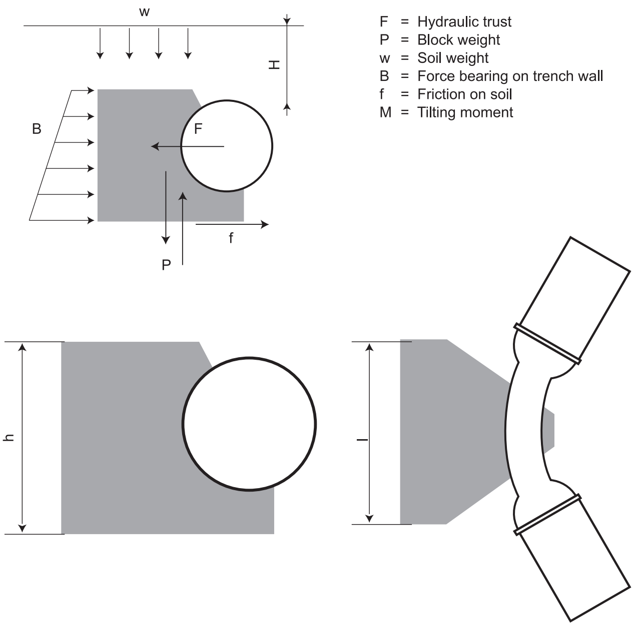


Figure 5.25. Anchorage of pipe bends.

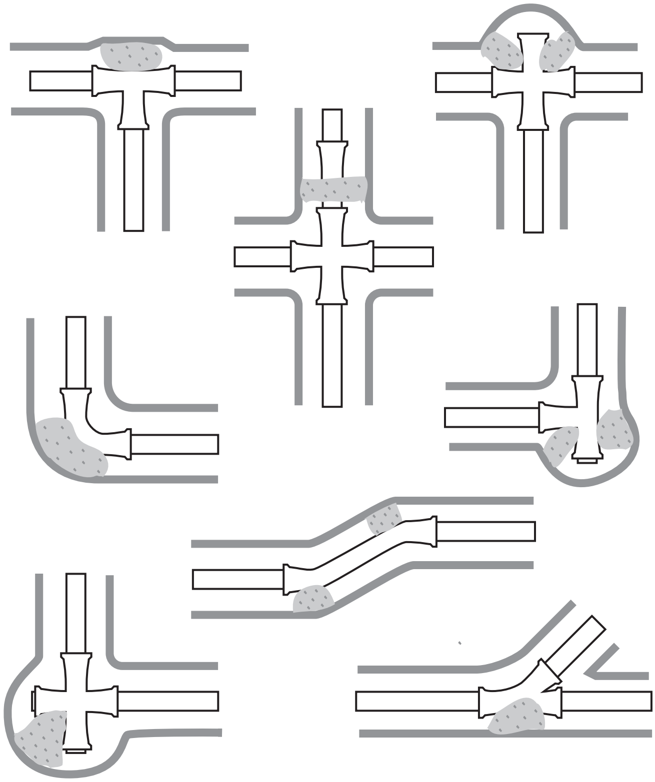


Figure 5.26. Thrust blocks in distribution systems (AWWA, 2003).

5.3.5 *Backfilling*

Backfilling of the trench can be done in two phases: partly, immediately after pipe laying to prevent floating caused by sudden heavy showers, and finally, after completion of the hydraulic tests. Two general layers can be distinguished (Figure 5.27):
1 pipe surround (initial backfill),
2 main backfill (infill).

The surround provides stability and protection for the pipe and increases the bearing capacity for external loads. The type of material used depends on the pipe characteristics and soil conditions. The infill varies according to the area involved and stability of the surface.

Fine material should always be used for the initial backfill; excavated sub-soil may also be suitable. Stones, rocks and any sharp materials are not allowed close to the pipe. The soil is normally placed in the trench in layers of 15–20 cm, and each layer is well compacted by machines that do not damage the pipe. The pipe can also be partly surrounded by the initial backfill but this reduces its supporting strength to a large extent; Table 5.2 illustrates this.

Top backfill in urban areas usually has to follow specifications required by road authorities, in open areas it is more related to aesthetics.

5.3.6 *Testing and disinfection*

As soon as the pipe laying is completed, a hydraulic test has to be carried out to check the quality of workmanship, namely
– the mechanical strength and leak tightness of the system,
– the strength of the anchorage and support structures.

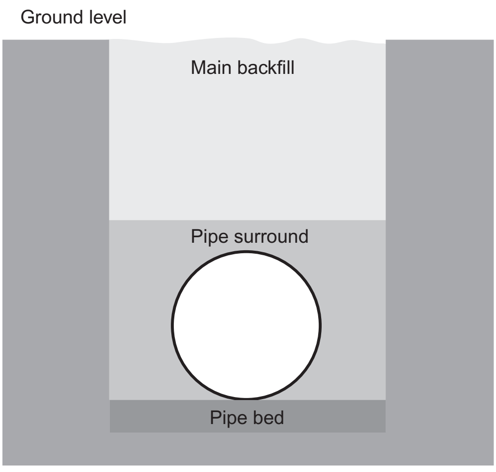


Figure 5.27. Pipe backfilling.

Table 5.2 Load-bearing strength of rigid pipes (AWWA, 2003).

Degree of initial backfill	Increase in load-bearing strength (%)
No initial backfill	–
Backfill up to 50% of horizontal diameter	36
Backfill up to 60% of horizontal diameter	73
Backfill up to full diameter (half pipe)	114
Backfill covering entire pipe (Figure 5.27)	150

All changes of directions, fittings and valves should be permanently anchored before the test starts. The ends of the tested section must be securely closed and temporarily anchored as well. There must be sufficient backfilling to prevent movement of the pipes during the test, but the joints should be left exposed until testing has been completed (Figure 5.28).

Water mains can be tested in lengths varying from a few hundred metres up to about a kilometre; although possible in theory, in practice it is more difficult to detect leaks with distances of more than 500 m. Pending good initial results, the length of the sections that are tested can be increased as the work progresses. The test pressure applied depends on the regulations. For distribution systems, it is usually 50% higher than the maximum working pressure. A common method, shown in Figure 5.29, is described in detail below.
The test starts by filling the section with chlorinated water, if possible from the lower of the two pipe ends. It is essential to ensure that the main has been completely purged of air before it is pressurised.

After filling, the section should be left under moderate pressure until stable conditions are achieved. The length of this period depends on the quantity of air trapped and the absorption of pipe material. For absorbent pipes such as AC and concrete, or cement-lined pipes, it can take a couple of days before the pipe material is fully saturated.

The pressure is then brought up to the test value by a hand-operated pump and all exposed parts of the section are examined for water tightness. The duration of the test and interpretation of the results depend on regulations. According to the French standards, the test is successful if the pressure in the section does not drop more than 2 mwc within 30 minutes (Pont-a-Mousson, 1992). By British standards, a leakage level in the section is monitored through the amounts of water pumped to re-establish the testing pressure after the drop. A tolerable leakage is 0.1 l/d per km of section and per mm of pipe diameter, under 30 mwc of pressure (Brandon, 1984).

If the limits are exceeded, a systematic search for leaks must be made. If standard methods of leak detection do not produce a result, the testing has to be repeated on shorter sections in order to isolate the

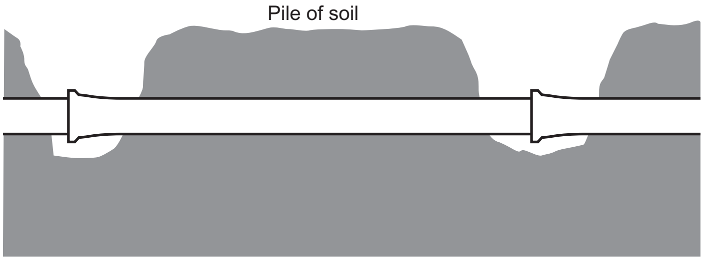


Figure 5.28. Preparation for pipe testing.

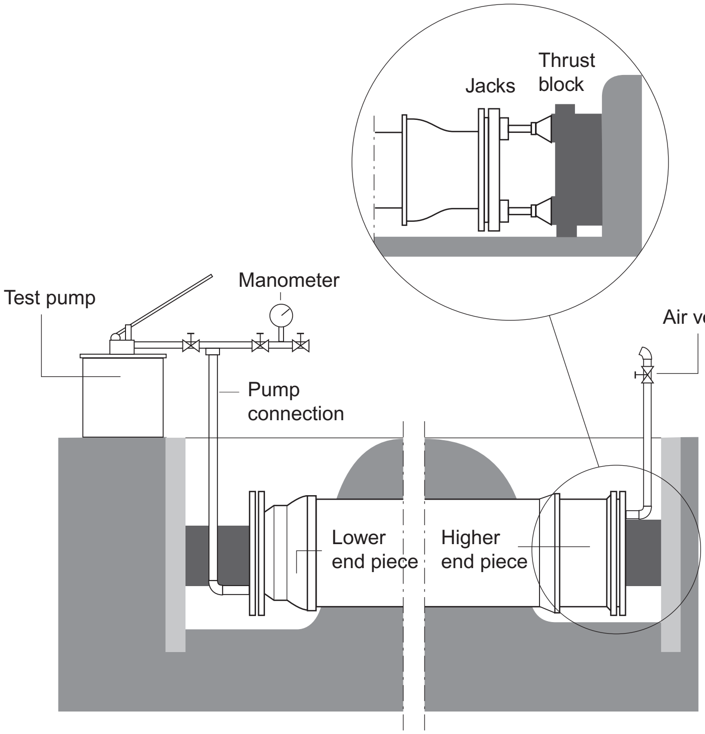


Figure 5.29. Pipe testing equipment (Pont-a-Mousson, 1992).

leakage points. On rare occasions air pressure testing can be used for locating defective joints in waterlogged conditions.

When the hydraulic test has been successfully completed, the pipeline should be flushed out to remove any remaining debris and properly disinfected. The British regulations prescribe a chlorine disinfection applied in a dose sufficient to maintain a residual of 20–30 mg/l, which must stand for at least 16 hours. Before being washed out, the water in the pipeline must be de-chlorinated. After washing out, the network can be charged by water, and after testing of the water quality, it can be put into service.

CHAPTER 6

Operation and Maintenance

6.1 NETWORK OPERATION

The consumer's requirements will not be satisfied in a poorly operated network, even if it has been well designed and constructed. Making errors in this phase amplifies the common problems and their implications that were already mentioned in previous chapters:
– low operating pressures causing inadequate supply,
– high operating pressures causing high leakage in the system,
– low velocities causing long retention of water in pipes and reservoirs,
– frequent changes of flow direction causing water turbidity.

These problems can have a serious impact on public health and coping with them also influences maintenance requirements and the overall exploitation costs.

The operation of gravity systems is rather simple and deals with the balance between supply and consumption, which can be controlled by operating valves. Pressure limitations in the gravity systems that result from topographic conditions become even bigger in the case of a bad design. A wrongly elevated tank, incorrect volume or badly sized pipe diameter will not guarantee optimal supply, and errors will have to be corrected by what would otherwise be unnecessary pumping.

In pumped systems, a more sophisticated operation has to be introduced to meet the demand variations and keep the pressures within an acceptable range. Computer simulations are an essential support in solving problems such as these. As well as pressures and flows, network models can process additional results relevant to the optimisation of the operation, such as: power consumption in pumping stations, demand deficit in the system, or decay/growth of constituents in the network. These models are also able to describe the patterns developed during irregular supply situations. Finally, the models can be linked with monitoring devices in the system, which enables the whole operation to be conducted from one central place.

An example in Figure 6.1 shows a comparison between computer model output and telemetry ('+' markers) for a pressure-controlled

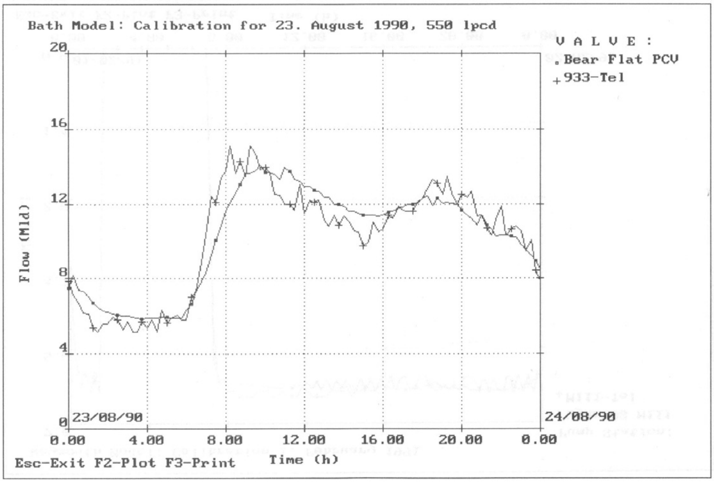


Figure 6.1. Fitting of the telemetry and computer model results (Obradović, 1991).

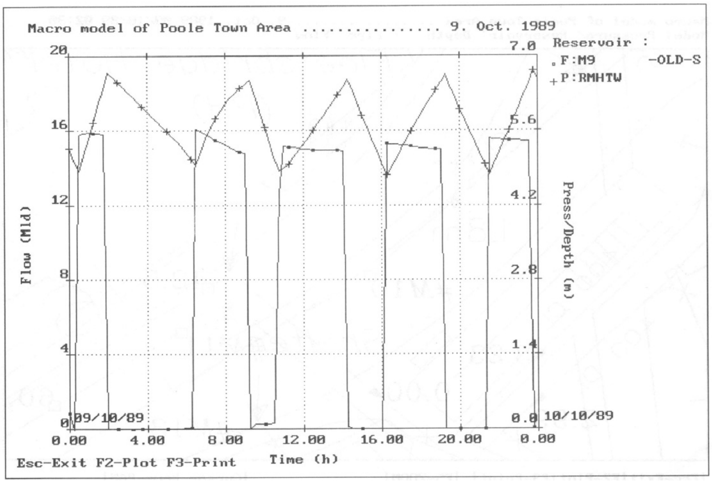


Figure 6.2. Simulated operation of a pumping station and corresponding water tower (Obradović, 1991).

valve. In Figure 6.2, another example of a pumping regime controlled automatically by the water level variation in a corresponding water tower is presented. This example indicates clearly the hydraulic link between the pumping station and the water tower that is filled with the pumped water until its maximum water level is reached; the pump is switched on again when the level in the water tower drops to the minimum.

Computerised operation does not necessarily require lots of expensive equipment when compared to the overall investment cost of the distribution network. Maintaining this equipment in good condition is more of a concern, especially if it operates under extreme temperatures, humidity, interrupted power supply, etc. Nevertheless, good knowledge about the hydraulic behaviour of the system combined with well-organised work by sufficiently trained personnel will help to save on both the investment and running costs.

6.1.1 *Monitoring*

Monitoring of water distribution systems provides vital information while setting up their operational regimes. It predominantly comprises:
– monitoring of pressure-, water level- and flow variations,
– monitoring of water quality parameters, such as temperature, pH, turbidity, chlorine concentration, etc.

Pressure-, level- and flow variations can be observed periodically for specific analyses (e.g. leakage surveys or the determination of a consumption pattern). When monitored continuously, they may indicate:
– operational problems that require urgent action (e.g. pressure drop due to a pipe burst),
– need for change in the mode of operation, as is the case in Figure 6.2.

Monitoring of water quality parameters can also help to detect inappropriate operational regimes. In addition, water quality parameters outside the normal range often indicate a need for necessary maintenance (illustrated later in this chapter). As with hydraulic measurements, the selection of sampling points should provide a good overview of the whole system, preferably at the source, reservoirs and other easily accessible locations where long retention times are expected.
Decisions on the spatial distribution of measuring points depend on the configuration of the system. Pressure- and flow- meters have to be installed in all the supply points and booster stations. Water levels in the reservoirs should also be permanently recorded. The measurements in main pipelines may be registered at critical points of the system (relevant junctions, extreme altitudes, pressure reducing valves, system ends, etc.).
All these data can be captured in one of the following ways:

Telemetry

1 *Telemetry* – There is a permanent online communication between the measuring device and control command centre where the parameter can be monitored round the clock.

Data loggers

2 *Data loggers* – Here a measuring device is permanently installed but the data for certain time intervals are captured periodically and will be processed and analysed later. Hence, the results of the measurements are not directly visible.

Local reading

3 *Local reading* – Direct readings can be obtained from the measuring devices' display and immediate action taken if required.

Sampling

4 *Sampling* – The water sample will be occasionally taken and analysed in a laboratory.

One example of permanent monitoring station installed on a transmission main is shown in Figure 6.3.

Figure 6.3. An on-line monitoring station at a fixed location.

Table 6.1. Data capturing methods and points (Obradović and Lonsdale, 1998).

Monitoring point	Flow	Water level	Pressure	Pump status	Pump speed	Valve opening	Volume	Chlorine	Turbidity
Water source	T/H	T/M							T/M
Well pumps	T/M			L/M	L/M				
Treatment plant	T/H	T/H						T/M	T/M
Main reservoir	T/M	T/H						L/H	
Main PST	T/H		L/H	T/L	T/M			L/H	
Local PST	T/H		L/H	T/L	L/M			L/M	
Service reservoir	T/M	T/H	L/H					L/M	
Booster PST	T/M			T/L	L/M			L/M	
Control valve	T/L		T/L			T/M			
Shut-off valve	L/H					L/H			
Distribution area	T/L		T/L				D/H		
Supply zone	D/H						D/M		
Demand district	D/H						D/L		
Control node	D/M		T/H					T/M	
Special customer	T/H						D/H	S/	
Large customer	D/H						D/M	S/	
Ordinary customer	D/M						D/L	S/	

X/Y: Method/Priority
X: T – Telemetry, D – Data loggers, L – Local instruments, S – Sampling
Y: H – High, M – Medium, L – Low

Table 6.1 gives the recommended order of priority in selecting an appropriate data capturing method in a water supply system.

Special conditions in the system can sometimes help to draw conclusions about its operation. The following example from The Netherlands (Cohen and Konijnenberg, 1994) shows the monitoring of the retention times by measuring natrium concentrations in different spots of the network. Retention times of up to 60 hours were observed in a distribution area near Amsterdam, during maintenance of its main softening installation (Figure 6.4). The installation was stopped for 48 hours, which caused a temporary drop in Natrium concentrations. Obviously, this

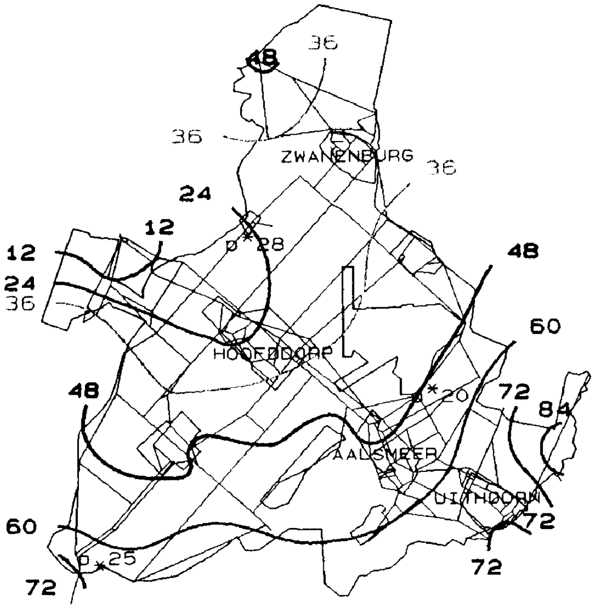


Figure 6.4. Retention times in a distribution network (Cohen and Konijnenberg, 1994).

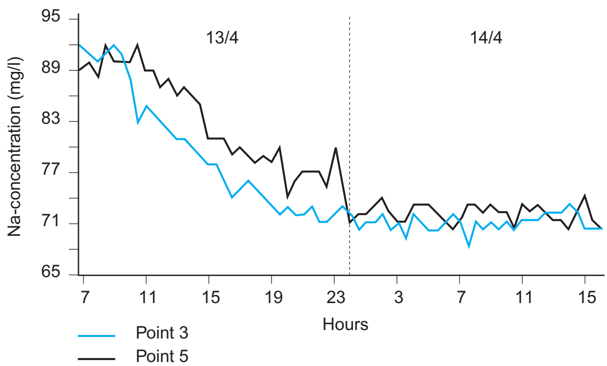


Figure 6.5. Drop in Na-concentration as a result of hard water in the system (Cohen and Konijnenberg, 1994).

effect could be registered sooner in the points located closer to the source (Figures 6.5 and 6.6).

6.1.2 *Network reliability*

A network is reliable if it can permanently perform in accordance with the design criteria. In reality, due to unforeseen events, this is never the case. It is therefore more realistic to define network reliability as *a probability of guaranteed minimum quantity, supplied in any (irregular) situation*.

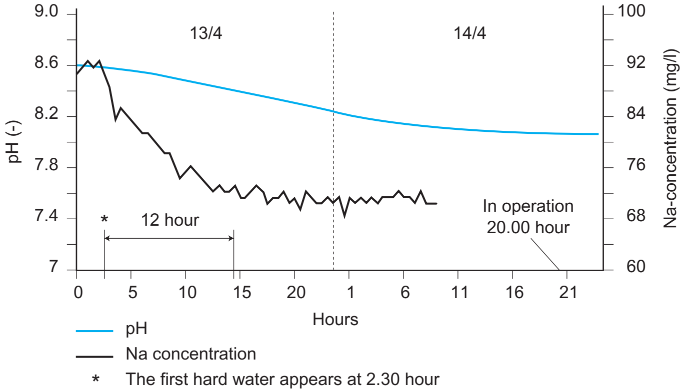


Figure 6.6. Comparison between Na-concentration and pH-values in the system (Cohen and Konijnenberg, 1994).

Failures
Calamities Disasters

When interruptions occur, consumers are normally not concerned with the cause, but rather with the consequences. Accordingly, the irregular events can be classified as *failures, calamities* or *disasters*. In the case of failures, a local interruption of the supply area will be caused. These are usually breaks in the distribution pipes that can be repaired within 24 hours. Failure of some major system component (a pumping station, or main transmission line) is considered as a calamity, which will affect a larger number of consumers and in most cases for more than 24 hours. An event involving the simultaneous failure of major components is treated as a disaster.

Temporary shortages of supply can appear due to:

– pipe breakage,
– power or mechanical failure in the pumping station,
– deterioration of the raw water quality (source),
– excessive demand in other parts of the network,
– maintenance or reconstruction of the system.

Pipe breakage is the most difficult to prevent because of the wide range of potential causes. Table 6.2 shows the statistics for some European cities (main pipes only). The data in it offer different pictures about leakages depending on the way they are presented. For instance, three times less bursts are observed in the network of Zurich in Switzerland compared to Vienna in Austria, while at the same time the number of leaks per 100 km of the network is higher in Zurich.

The bursts occur more often with smaller pipes and service connections, but create a rather insignificant impact on the overall water loss and hydraulic behaviour of the system. Reasonably accurate predictions can be derived from local observations of the event occurrence. The type of relation between the number of bursts and pipe diameters will be as shown in Figure 6.7 in most cases.

Table 6.2. Pipe burst occurrences (Coe, 1978).

City, Country	Total bursts per annum	Bursts per 100 km pipes
Vienna, Austria	1700	47
Salzburg, Austria	83	18
Antwerp, Belgium	225	13
Copenhagen, Denmark	100	12
Helsinki, Finland	116	14
Frankfurt, Germany	600	40
Rotterdam, The Netherlands	400	19
Oslo, Norway	320	27
Barcelona, Spain	2850	115
Zurich, Switzerland	550	56

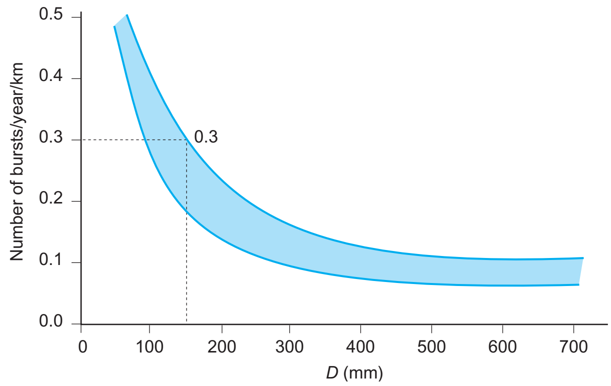


Figure 6.7. Bursting frequency of pipes of various diameters.

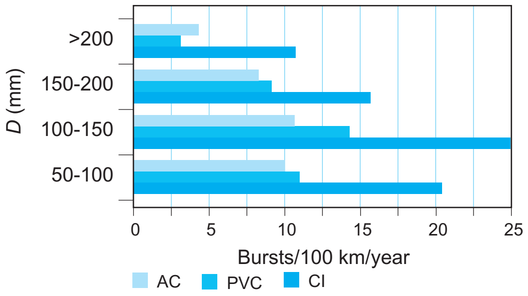


Figure 6.8. Average frequency of pipe bursts in The Netherlands (Vreeburg *et al.*, 1994).

Concerning the type of materials, in general, corrosion-attacked pipes are the least reliable. However, these experiences are not transferable in practice, and keeping local records about the system failure is an essential element of reliability analyses. The records for the three types of pipe materials mostly used in The Netherlands are given in Figure 6.8.

According to this diagram, CI pipes appear to be the most vulnerable; in this particular case for well-known reasons. These were the first generation pipes (laid before ±70 years), while AC and PVC pipes belong to the second and third generation of the twentieth century (±40 and 20 years old, respectively). Apart from these pipes, experience with the latest materials (PE, GRP) is too short to draw firm conclusions yet.

A simplified method for assessing the network reliability is based on the following formula:

$$R = 1 - \frac{Q_o - Q_f}{Q_o} \tag{6.1}$$

where Q_f represents the available demand in the system after the failure, against the original demand Q_o. The effects of the failure expressed in reduction of supply can be foreseen by running computer simulations. This is normally done for maximum supply conditions and without selected components in operation. The assessment requires repetitive calculations but apart from that, the results can accurately point out weak points in the system. The burst of a pipe carrying large flows always has more far-reaching adverse consequences on the pressures and flows in the system than the burst of some small or peripheral pipe. For a more complex consideration of reliability, the failure frequency and average time necessary for repair may also be included.

Hydraulic reliability

A practical method of this type is suggested by Cullinane (1989), who defines the nodal reliability as a percentage of time in which the pressure at the node is above the defined threshold. It is known as the *hydraulic reliability* and reads as follows:

$$R_j = \sum_{i=1}^{k} \frac{r_{ij} t_i}{T} \tag{6.2}$$

where R_j is the hydraulic reliability of node j, r_{ij} is the hydraulic reliability of node j during time step i, t_i is the duration of time step i, k is the total number of the time steps, T is the length of the simulation period. Factor r_{ij} takes value 1 for the nodal pressure p_{ij} equal or above the threshold pressure $p_{\min}$, and $r_{ij} = 0$ in case of $p_{ij} < p_{\min}$. For equal time intervals, $t_i = T/k$.

The reliability of the entire system consisting of n nodes can be defined as the average of all nodal reliabilities:

$$R = \sum_{j=1}^{n} \frac{R_j}{n} \tag{6.3}$$

The above equations assume that all network components are fully functional, which is rarely the case. The expected value of the nodal reliability can be determined as:

$$RE_{jm} = A_m R_{jm} + U_m R_j \qquad (6.4)$$

where RE_{jm} is the expected value of the nodal reliability while considering pipe m, A_m is the availability of pipe m, i.e. the probability that this pipe is operational, U_m is the unavailability of pipe m, $U_m = 1 - A_m$, R_{jm} is the reliability of node j if link m is available, i.e. in operation and R_j is the reliability of node j if link m is not available, i.e. not in operation. The component availability can be calculated on an annual basis from the following equation:

$$A_m = \frac{8760 - \mathrm{CMT} - \mathrm{PMT}}{8760} \qquad (6.5)$$

where CMT represents the annual corrective maintenance time in hours and PMT is the annual preventive maintenance time in hours. These figures should be available from the water company records.

The values of R_{jm} and R_j in Equation 6.4 are determined from Equation 6.2, running the network computer simulation once with the link m operational and then again, by excluding it from the layout.

A single transportation pipe has practically no reliability as any burst will likely result in a severe drop in supply and pressures; during repair all downstream users will have to be temporarily disconnected (Figure 6.9).

A burst in the case of parallel pipes causes a flow reduction dependant on the capacity of the pipe/pumping station remaining in operation,

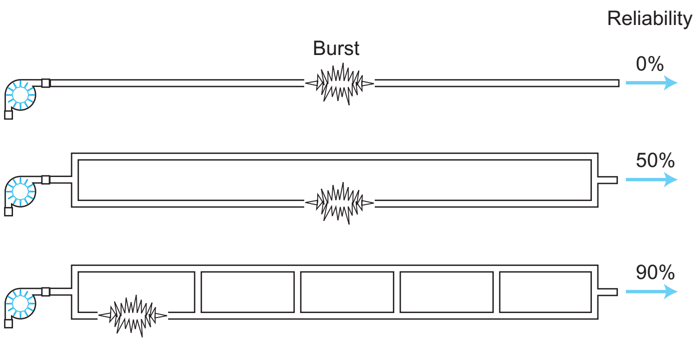


Figure 6.9. Reliability assessment.

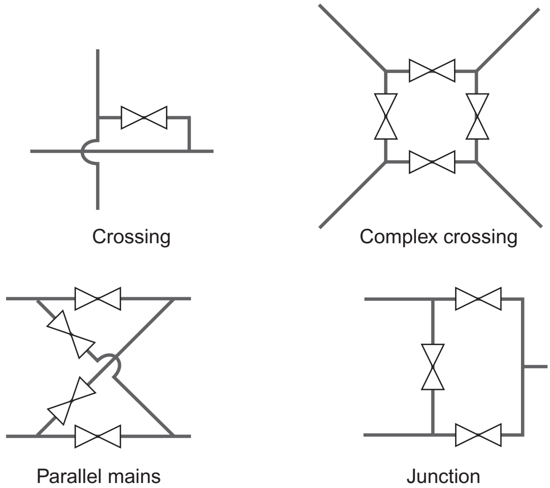


Figure 6.10. Technical provisions for improvement of reliability (van der Zwan and Blokland, 1989).

say 50%. Further improvement of the reliability will be achieved by introducing the following technical provisions:
- parallel pipes, pipes in loops, cross connections,
- pump operation with more units,
- alternative source of water,
- alternative power supply,
- proper valve locations,
- pumping stations and storage connected with more than one pipe to the system,
- reservoirs with more compartments,
- bypass pipes around the pump stations and storage etc.

A few examples of possible cross-connections are shown in Figure 6.10. In long transmission lines, these are usually constructed every 4–5 km, in distribution mains every 300–500 m and in rural areas every 1–2 km.

Setting the standards in technical measures that can improve the network reliability is rather difficult due to the variety of situations and consequences that can occur. Nevertheless, some guidelines may be formulated if there are more serious failures. For instance, the Dutch Waterworks Association (VEWIN) proposes 75% of the maximum daily quantity as an acceptable minimum supply in irregular situations. This should be applicable for a district area of ±2000 connections. Within such an area, valves should be planned to isolate smaller sections of 10–150 connections, when necessary.

6.1.3 *Unaccounted-for water and leakage*

Unaccounted-for water

The charged water quantity will always be smaller than the supplied amount. Moreover, the volume of water actually consumed is also smaller than the supplied amount, be it charged or not. The difference in the first case refers to the *unaccounted-for water* (UFW) while the second one represents leakage. Leakage is usually a major factor of UFW. Other important factors can be faulty water meters, illegal connections, the poor education of consumers etc.

Non-revenue water

In more recent terminology, the difference between the supplied and the charged quantity is defined as *non-revenue water* (NRW), whilst the unaccounted-for water is the part of NRW that remains after deducting unbilled but authorised consumption. Examples of such consumption are the water used for backwashing of filters, flushing of pipes, washing streets, fire fighting, public taps and fountains, parks etc. The volume used for these purposes is usually marginal compared to the total water supplied, which makes the difference between UFW and NRW small in many systems. A structure of the total system input volume is shown in Table 6.3. For more details on the terminology see IWA (2000).

Magnitude and causes

There are two usual ways of expressing unaccounted-for water:

1 as a percentage of (annual) water production,
2 as a specific value, in m^3/d per km length of the network.

Comparison between these two approaches may lead to conflicting conclusions, as the example of records for a number of countries in Figure 6.11 shows. The gross UFW percentage does not necessarily coincide with the UFW quantities spread over the length of the network.

Table 6.3. Structure of a water supply system input volume (IWA, 2000).

Authorised consumption	Billed authorized consumption	Billed metered consumption (including exported water)	Revenue water
		Billed unmetered consumption	
	Unbilled authorized consumption	Unbilled metered consumption	Non-revenue water
		Unbilled unmetered consumption	
Water losses (UFW)	Apparent losses (Commercial losses)	Unauthorised consumption	
		Metering inaccuracies	
	Real losses (Physical losses)	Leakage in transmission and distribution lines	
		Leakage and overflows at storage tanks	
		Leakage on service connections up to customer meters	

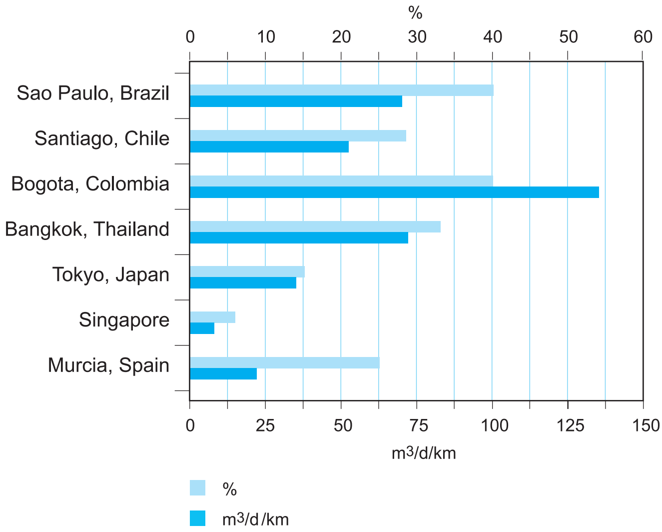


Figure 6.11. UFW statistics (World Bank, 1996).

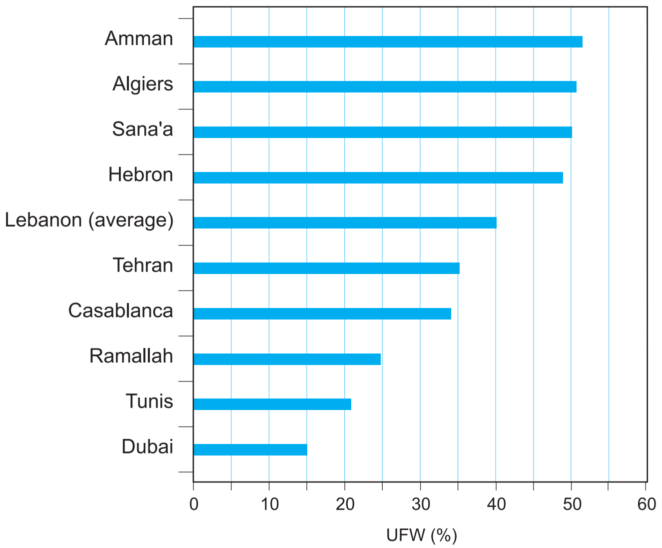


Figure 6.12. UFW in some countries of the MENA Region (World Bank, 2000).

The statistics on UFW show very high figures for many cities in the developing countries where water is often scarce. The examples are shown in Figures 6.12 and 6.13.

Besides leakage, a significant impact on UFW in the developing countries frequently comes from the malfunctioning of water meters, their inaccurate and irregular reading, or from illegal connections. All these contribute to the high UFW levels, as shown in Table 6.4 (Thiadens, 1996).

Similar figures in the developed world are much lower. Typically, the UFW levels in the Western Europe are between 5% and 15%.

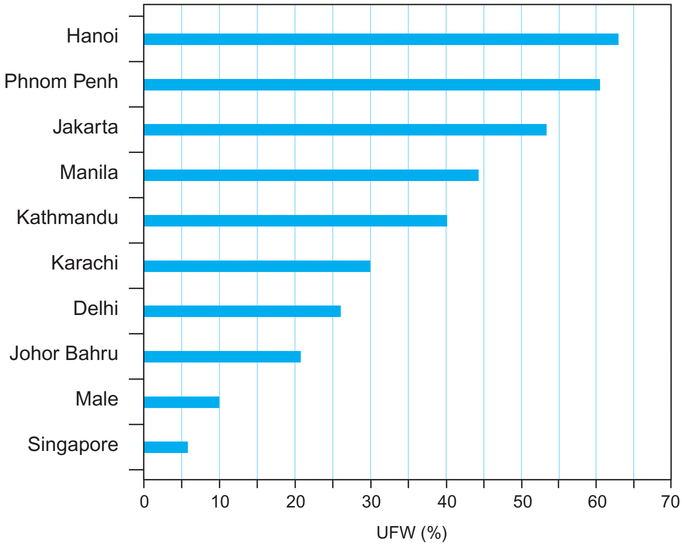


Figure 6.13. UFW in some Asian cities (ADB, 1997).

Table 6.4. Main components of UFW in developing countries (Thiadens, 1996).

UFW components		Bandung (Indonesia)	Chonburi (Thailand)	Petaling Jaya (Malaysia)
Physical losses (%)	Trunk mains, distribution system	21	2	2
	Service connections	10	34	17
Non-physical losses (%)	Illegal connections	6	2	2
	Meter under-registering and billing	6	8	15
Total UFW		43	46	36

The breakdown of the volume distributed in the city of Nuremberg in Germany is given in Table 6.5 (Hirner, 1997).

In numbers, the leaks most often occur at service connections. However, these result in small water losses, eventually paid for by consumers in many cases. For that reason, water companies are more concerned by losses in the distribution system that may pass unattended for a long period of time.

Common factors influencing leakage can be split in two groups. The first deals with soil characteristics and the corresponding human activities; the main factors in this group are:

– soil movement and aggressiveness,
– heavy traffic loadings,

Table 6.5. The breakdown of the volume distributed in Nuremberg, Germany (Hirner, 1997).

Accounted for water – 91.3 (%)	Charged water – 84.8	
	Bulk supply water – 6.2	
	Public use (park, fountains, etc.) – 0.3	
Unaccounted for water – 8.7 (%)	Apparent losses – 3.5	Unmetered usage – 0.5
		Own waterworks consumption – 1
		Meter errors – 2
	Real losses – 5.2	Pipe breaks – 3.5
		House connections – 1.7

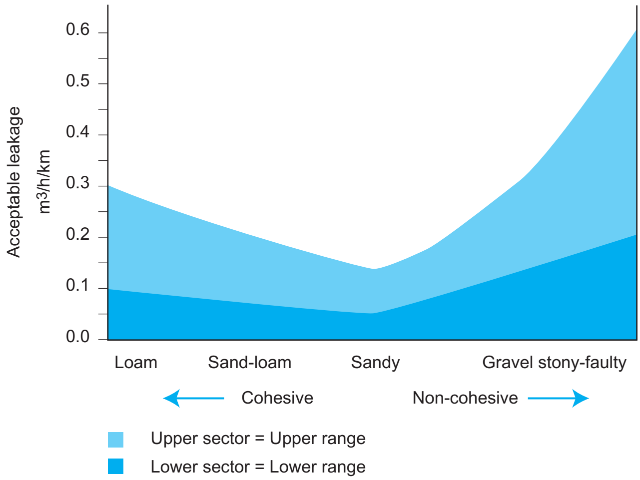


Figure 6.14. Leakage in various soil types (Weimer, 1992).

– damage due to excavations,
– damage due to the growing roots of plants.

The second group of factors deals with the system components, its construction and operation. Here, the main factors are:
– pipe age, corrosion, and defects in production,
– high water pressure in the pipes,
– extreme ambient (winter) temperatures,
– poor quality of joints,
– poor quality of workmanship.

In many cases the leaks are related to the type of soil. In that respect, the most favourable conditions will be met in sandy soils. Figure 6.14 shows the German experience of acceptable levels of leakage in various types of soils (Weimer, 1992).

Leak detection methods
Global estimates of leakage come from an annual balance of the delivery and metered consumption for the whole network. Bursts of main

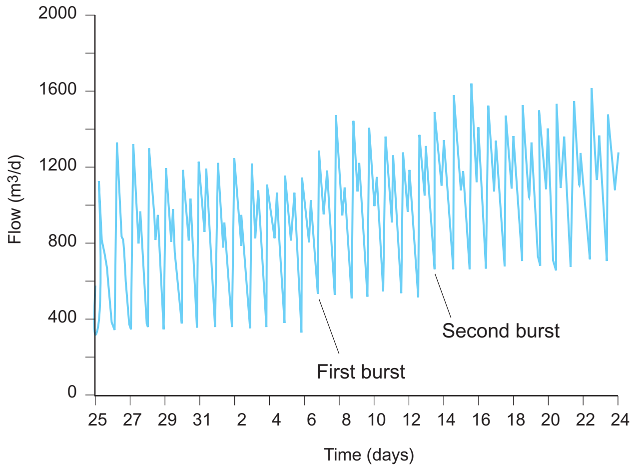


Figure 6.15. Leak detection from the demand monitoring.

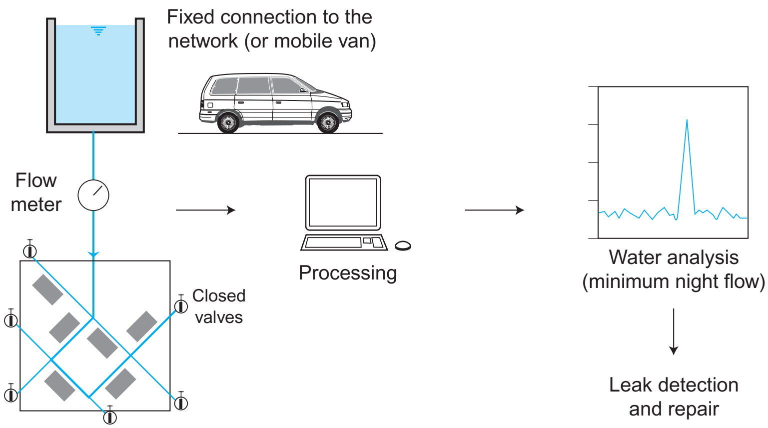


Figure 6.16. District monitoring of night flow.

pipes can be detected by the flow measurements at supply points (Figure 6.15).

The earlier information is not based on specific monitoring of leaks. To enable leak detection, parts of the system have to be inspected over a period of several hours or days, depending on the size of the district. These temporary measurements are usually carried out overnight, when real consumption and overall noise level are at a minimum. A flow meter can be permanently installed, or a van with the equipment is brought to the location (see Figure 6.16). The area will be isolated from the rest of the system by closing the district valves and its in- and out-flow will be measured. The company would normally possess some records on the night consumption in the area. Everything that is detected in addition to that is a form of UFW, mostly leakage.

The measurements can be repeated in weekly intervals throughout a period of a few months. Possible pipe burst between the two measurements should be reflected in a sudden increase in registered demand.

The average leakage level can also be estimated by measuring pressures in the system, once the relation between the pressure and night flow has been established. The method shown in Figure 6.17 is applied in England (Brandon, 1984). The example in the figure shows how reducing the pressure from 70 to 38 mwc cuts the leakage to half of the existing level.

Flow and pressure measurements do not indicate the exact location of leaks. In the case of severe breaks, water may appear on the surface, but more often leak detection techniques have to be applied. The most popular are:

1 acoustic (sound) method,
2 leak noise correlation,
3 tracer techniques.

Acoustic detectors

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 the stick, to the amplifier connected with the stethoscope (Figure 6.18). This method is not always reliable due to the fact that some leaks produce undetectable noise. Moreover, locating the pipe route is much more difficult in the case of plastic and concrete pipes than in the case of metal pipes, as already

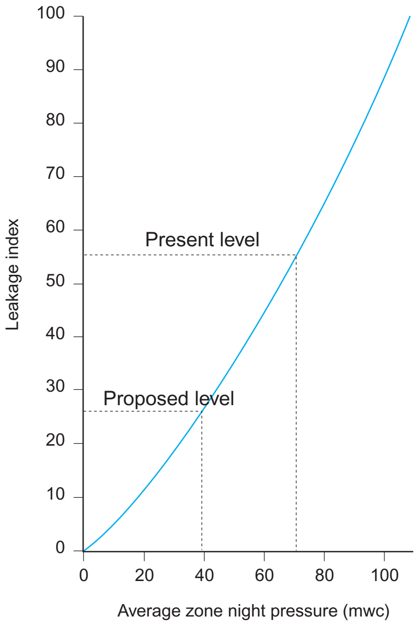


Figure 6.17. Pressure-leakage relation (Brandon, 1984).

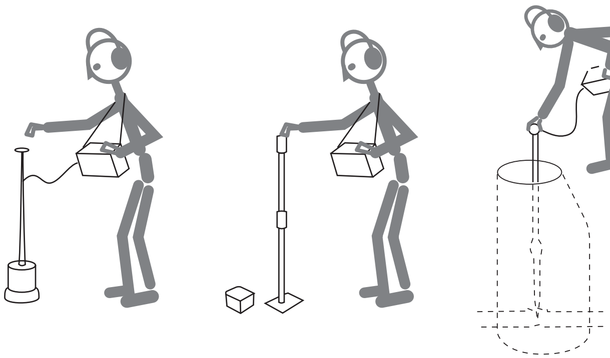


Figure 6.18. Leak detection by the acoustic method.

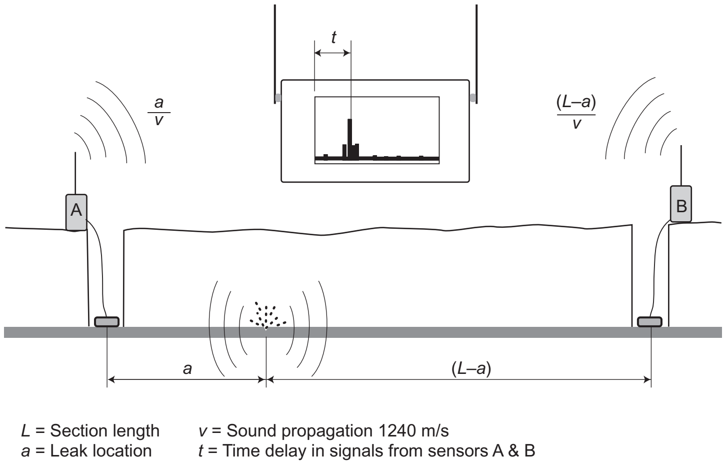


Figure 6.19. Correlation method – principle.

shown in Tables 4.6. and 4.7 Nevertheless, with skilled personnel working under silent (night) conditions, most of the leaks in metal pipes can be discovered.

Correlation method

Another method based on sound detection is the *correlation method* (Figures 6.19 and 6.20). This method uses the constant sound propagation in water, which happens at a speed of $\pm$1240 m/s, for detection of leakage. By placing the microphones at the ends of the controlled section, the difference (t) in time required for the leak noise to reach the microphones can be measured by a device called a *correlator*. For the known length L of the section, position a of the leak can be calculated. This method is fairly effective in detecting leaks under high background noise levels and can therefore be applied during the daytime. As in the case of acoustic detectors, it might be less successful in the case of non-metal pipes or if more than one leak exists along the inspected pipe route.

Correlator

Figure 6.20. Correlation method – equipment (Manufacturer: Biwater-Spetrascan).

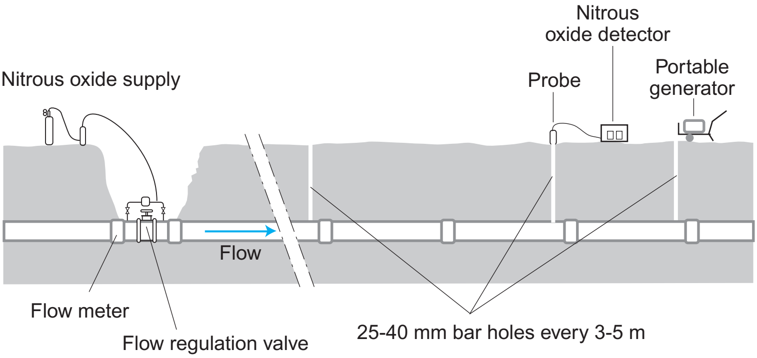


Figure 6.21. The tracer method.

Apart from ambient noise and pipe material, other factors generally affecting acoustic leak detection are low water pressure, variations in pipe depth or properties of bedding material, mixture of different soils and a high groundwater table.

Tracer method

The *tracer method* involves a gas being pressure-injected into the main that is under inspection; this main will be isolated from the rest of the system for this purpose. As water leaks, its pressure reduces to atmospheric level and the gas comes out of the solution. The presence of the gas is then tested by using the gas detector probe inserted into the holes made along the known pipe route (Figure 6.21).

A gas frequently used as a tracer is nitrous oxide (N_2O), being non-reactive, non-toxic, odourless and tasteless. It is soluble in water and can be registered in very small concentrations. Other gases can also be used, e.g. sulphur hexafluoride (SF_6). Work with gas tracers in US includes the inspection of empty pipes that can be pressurised by helium or nitrogen. Depending on the conditions, drilling of the probes might be avoided in this case (Smith *et al*., 2000).

The tracer methods are generally more expensive than the acoustic methods. Their advantage is that they are not dependent on the conditions required for acoustic leak detection. Furthermore, they can be used for locating bursts in empty pipes, which may sometimes be required for emergency reasons.

Organisation of the leak survey programme

There are various levels of treating the leakage problems. For leakage levels above ±30%, organisation of the leak survey and metering programme (LSM) is usually justified in the savings obtained. Below that level, an economic study should be carried out taking into consideration the costs of production, distribution and leakage control. The conditions of water sources are also a relevant element for the final decision. At the very least, by reducing the leakage level, an investment in the system extension can be postponed by several years (Figure 6.22).

Figure 6.23 shows the reduction of leakage applying different levels of control (Brandon, 1984). By the passive method, only the major leaks and leaks being reported by the consumers are repaired: the reactive approach, in which no systematic effort is made to measure or detect leakage.

Regular sounding is not considered to be the main component of the leakage detection procedure. When it is not selective, this method

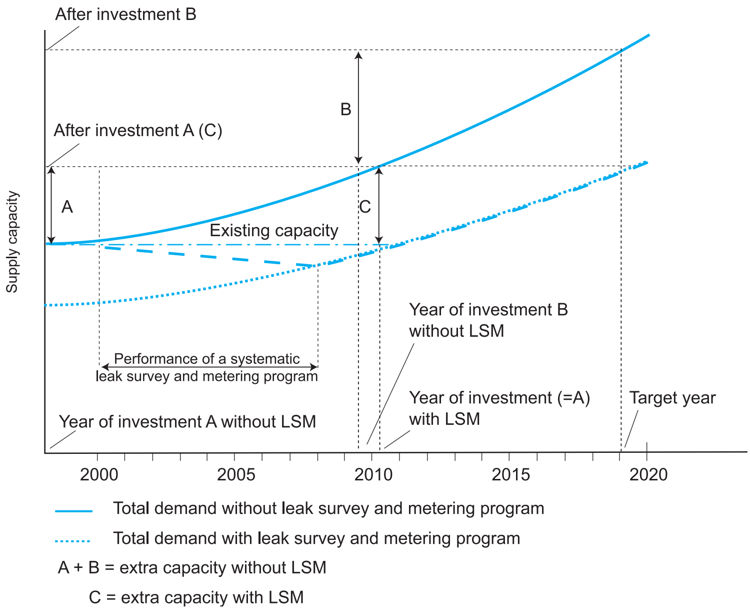


Figure 6.22. Savings obtained by the leakage reduction.

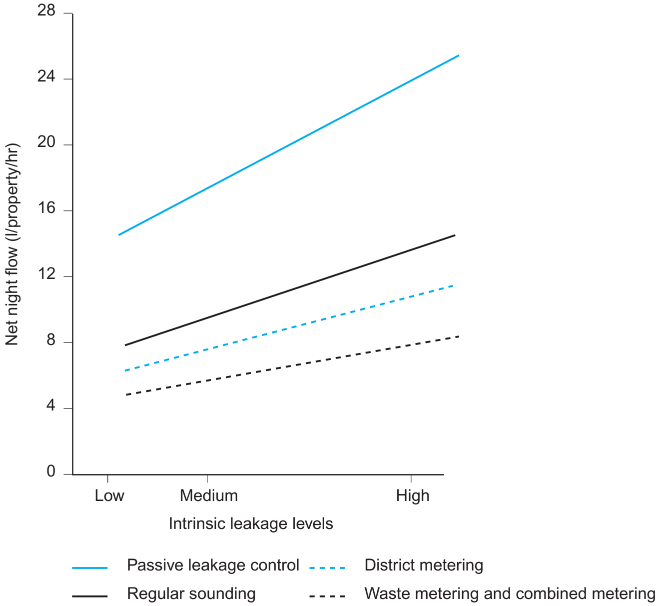


Figure 6.23. Levels of leak control.

requires a lot of manpower with sometimes unreliable results. Therefore, it is better to apply sounding techniques at a local level where leakage has already been indicated by measurements.

The most efficient (and expensive) approach includes prevention and reaction. Once the decision has been taken about systematic leak detection, a good organisation of the activities is required. A good organisational setup of a leak detection team in a water supply company can be as shown in Figure 6.24.

The main components of the leakage detection programme are:

1 data collection,
2 planning,
3 organisation,
4 leak survey and repair.

The data collection comprises information about:

– pipes: route, material, dimensions, age,
– measuring devices,
– valves and fire hydrants,
– service connections.

After this has been completed and evaluated, the next step is planning, comprising:

– preparation of operational charts for specific activities,
– selection of type and quantity of the equipment needed,
– planning of a training programme for personnel,
– determination of the priorities among the areas surveyed.

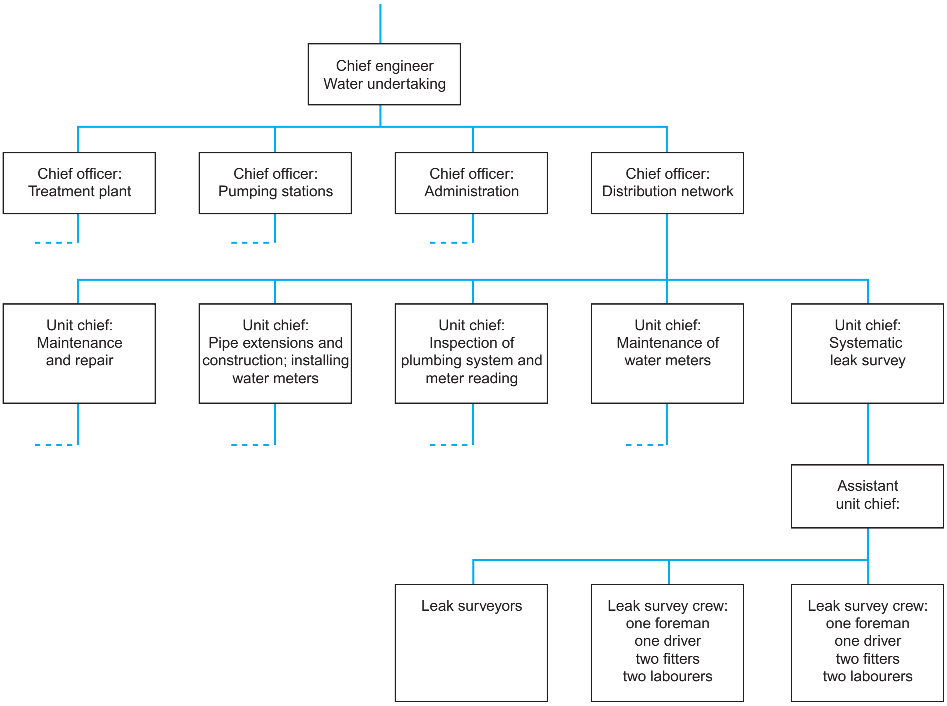


Figure 6.24. Leak detection team setup (Vreeburg and van den Hoven, 1994).

The organisational stage consists of:
– selection and training of the personnel,
– procurement of the equipment.

Finally, the leak survey can commence. The economic viability of the leakage survey programmes is evaluated based on:
– total amount of water loss due to leakage,
– minimum acceptable leakage percentage,
– total cost of water production,
– maximum saving if the minimum acceptable leakage can be achieved,
– investment and labour costs involved in the leakage detection programme,
– maintenance of the system at the minimum-acceptable leakage,
– the amount of water saved by the programme.

Water meter under-reading
If not regularly maintained and replaced, water meters may register inaccurate amounts of water. According to statistics, water meter under-reading is the second main source of UFW after leakage in the vast majority of countries.

The accuracy of water meters reduces after a few years in operation (see Figure 6.25). A severe drop in the accuracy of the measurements can occur as soon as five years of operation. This is particularly emphasised in the lower range of flow rates. The lack of accuracy of water meters may cause serious revenue losses for water companies. Therefore, the measuring devices have to be regularly controlled, and if necessary repaired and re-installed (the average cost in The Netherlands is 10–15 US$ per piece),

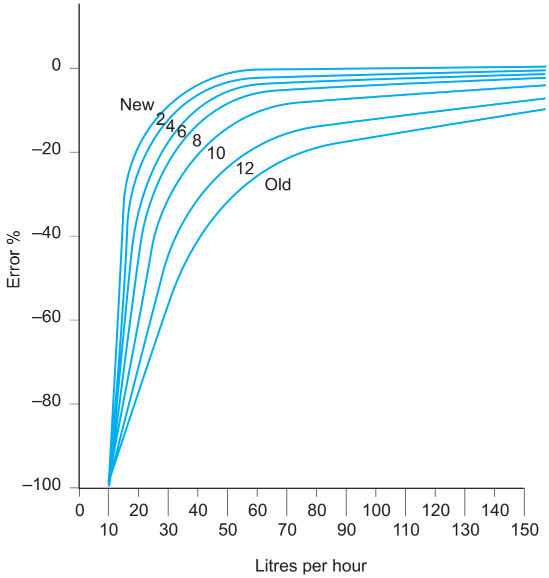


Figure 6.25. Drop in accuracy of water meters in operation.

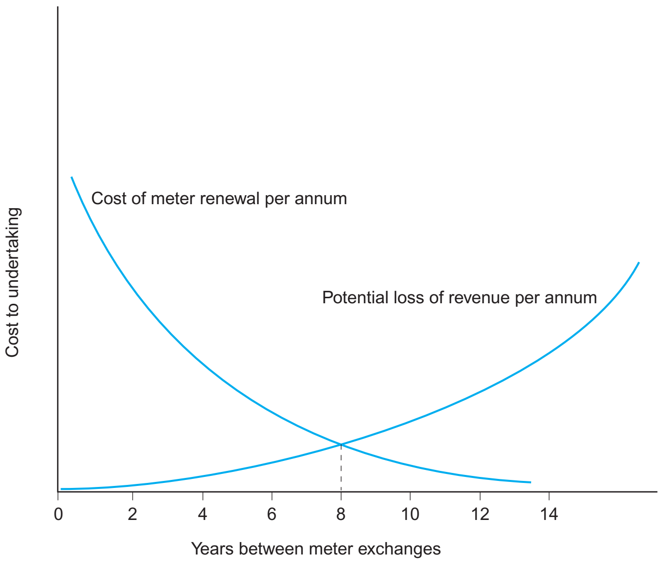


Figure 6.26. Economics of meter exchange periods.

or renewed (US$ 30–35 per piece). The choice between the two options will depend on the cost evaluation of each renewal and increased water loss due to malfunctioning (Figure 6.26). The economic period varies between 6 and 12 years and in exceptional cases longer; most commonly water meters are replaced every 8–10 years.

Water meters are sometimes installed within the premises. The fact that the real amount of water used is paid for increases awareness of the consumers. However, this principle is not always economical. It requires additional investment in equipment and personnel involved in installation, maintenance, replacement, reading and administration. The final decision normally depends on local conditions such as: average water demand, network coverage, labour costs, ability of the consumers to pay the bill, etc. Usually, the cost of individual metering is born fully or partly by the consumers.

6.1.4 *Corrosion*

Corrosion causes deterioration of material properties due to reaction with its environment. In water transport and distribution, this process takes place predominantly attacking pipes and joints, defined as:
– external corrosion, in reaction with the soil,
– internal corrosion, in reaction with water.

Concerning the materials, two types of corrosion can be distinguished:
1 metallic corrosion,
2 corrosion of cement-based products.

Metallic corrosion is a chemical reaction caused by the transfer of electrons (Figure 6.27). After a metal ion leaves the pipe surface (anodic site) and enters water, excess electrons migrate through the metal to a cathodic site where they are used by a balancing reaction. Three types of reactions are possible:
1 hydrogen evolution – typical in aggressive waters (with low pH),
2 oxygen reduction – typical in normal waters,
3 sulphate reduction – typical for the anaerobic conditions occurring in soils.

The direct consequence is that the pipe will lose its mass at the anodic site, which will be partly dissolved and partly accumulated at the cathodic site. Practical problems resulting from this are:
– loss of water and pressure due to leakages,
– increased pumping costs due to pipe clogging,
– malfunctioning of appurtenances in the system,
– malfunctioning of indoor installations,
– the appearance of bad taste, odour and colour in the water.

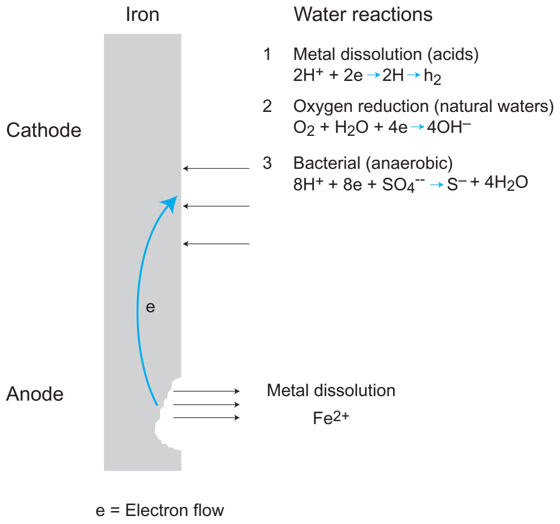


Figure 6.27. Metallic corrosion.

The corrosion of cement-based (or lined) materials is a chemical reaction in which the cement is dissolved due to the leaching of calcium at low pH values (in principle less than 6.0). This can be a problem in the case of cement-lined metal pipes but also with concrete or AC pipes, where the structural strength of the pipe may be lost or a metal (reinforcement) can be exposed to the water enhancing the corrosion process even further.

Corrosion forms
The actual mechanisms of corrosion are usually the interrelation of physical, chemical and biological reactions. Common forms of pipe corrosion are (AWWA, 1986):

Galvanic corrosion

Galvanic corrosion: When two different pipe metals are connected, the cathodic site tends to be localised on the less reactive material, and the anodic on the more reactive, causing corrosion known as galvanic corrosion. This kind of corrosion is typical at joints of indoor installations (e.g. copper pipe connected to galvanised iron causes corrosion of iron). The galvanic corrosion can be particularly severe at elbows.

Pitting

Pitting: This is localised, non-uniform corrosion hardly detectible before a hole appears. This is a potentially dangerous form of corrosion because even small holes may cause rapid pipe failure. Surface imperfections, scratches or deposits are favourable places for pitting corrosion.

Tuberculation

Tuberculation: Tuberculation occurs when pitting corrosion builds up at the anode next to the pit. Tuberculation rarely affects the water quality

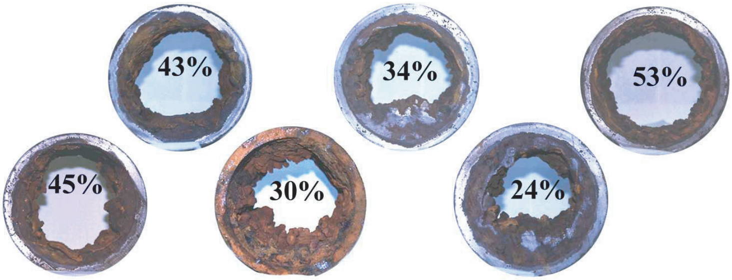


Figure 6.28. Effects of tuberculation on the reduction of the pipe cross-section (Courtesy Prof. V.L. Snoeyink, University of Illinois).

unless some of the tubercles are broken due to sudden changes in flow. Serious forms of this corrosion would lead to a drastic increase of pipe roughness, i.e. reduction in the inner diameter. Hence, this type of corrosion can be suspected by monitoring the hydraulic performance of the pipe. An example in Figure 6.28 shows the effects of tuberculation on the reduction of the cross section area of the CI pipes in the same distribution network. The percentage indicates the remaining area of the pipe cross-sections.

Crevice corrosion

Crevice corrosion: A form of localised corrosion usually caused by changes in acidity, oxygen depletion, dissolved ions etc. Crevices appear at joints or surface deposits.

Erosive corrosion

Erosive corrosion: By this corrosion a protective coating against corrosive attack is mechanically removed due to high velocities, turbulence or sudden changes in flow direction. Pieces of the pipe material can be removed, as well. This corrosion form is common at sharp bends.

Cavitation corrosion

Cavitation corrosion: This is a type of erosion corrosion caused by the collapse of vapour bubbles (most often at pump impellers, as explained in Chapter 4). It occurs at high flow velocities immediately following a constriction or a sudden change in direction.

Biological corrosion

Biological corrosion: The reaction between the pipe material and micro organisms that appear in pipes results in this form of corrosion. Biological corrosion is common in stagnant waters and at dead ends of networks. It is an important factor in the taste and odour problems that develop, but also in the degradation of the material. Control of biological growth is very difficult because it can appear in many protected areas (e.g. in accumulations of corrosion products) where disinfection by chlorine or oxygen is inefficient.

Corrosion and water composition

Water composition is a key factor that influences internal pipe corrosion. Much effort has been put into establishing the quantitative relationship

Table 6.6. Factors influencing the internal corrosion of metal pipe (Smith *et al.*, 2000).

Factor	Effect
pH	Low pH increases corrosion rate. High pH tends to protect iron pipe but may cause removal of zinc from brass and damage of copper pipe.
Dissolved oxygen	Increases the rate of many corrosive reactions.
Temperature	Increases the rate of many corrosive reactions. Decreases calcium carbonate solubility. Increases biological activity.
High flow velocity or turbulence	Increases potential for erosive corrosion.
Low flow velocity	Increases potential for crevice and pitting corrosion.
Alkalinity	Helps to form a protective coating of scale and buffer pH changes. High alkalinity increases corrosion of copper, lead and zinc.
Calcium hardness	Helps to form a protective coating of scale but at high concentration may cause tuberculation or excessive scaling.
Chlorine residual	Increases most types of corrosion of metal pipe. Decreases biological corrosion.
Chloride and sulphate	Increases corrosion of iron, copper and galvanised steel. Increases tendency for pitting corrosion.
Hydrogen sulphide	Increases corrosion.
Ammonia	Increases corrosion of copper pipe.
Magnesium	May inhibit precipitation of the calcite form of calcium carbonate and favours the formation of the more suitable aragonite form.
Total dissolved solids	Increases water conductivity, which tends to increase the corrosion rate.

between the water composition and types of corrosion. Factors influencing the internal corrosion of metal pipes, according to US experience, are listed in Table 6.6.

Specific research, which also covered some non-metal pipes, was carried out in The Netherlands (van den Hoven and van Eekeren, 1988). The impact of the following parameters was analysed:

1 pH value,
2 TIC (total inorganic carbon) as the sum of carbon based elements: (H_2CO_3), (HCO_3^-), (CO_3^{2-}),
3 SI (saturation index) as an indicator of water aggressiveness with respect to calcium carbonate: $(CaCO_3)$,
4 chloride and sulphate: (Cl^-), (SO_4^{2-}).

The findings per pipe material are listed as follows.

AC, concrete and cement mortar: Corrosion of cement predominantly caused by acids may result in:

1 disintegration of the inner surface layer of AC pipes and exposure of fibres that might easily be released into the water,
2 increase of pH (especially with water stagnation) and suspended solids, calcium, iron, aluminium and silicates,
3 reduction in pipe strength,
4 increased energy loss by increased wall roughness.

Present knowledge indicates that corrosion of cement-based materials and the release of asbestos fibres are acceptable when $SI > -0.2$ (minimal corrosion is obtained for positive SI-values). However, it is still doubtful whether the saturation index can be assumed to be a sufficient indicator of corrosion in this case.

Ductile iron and steel: Corrosion of these materials is predominantly caused by dissolved oxygen. Other components such as nitrate, hydrogen ions and some chlorine components (where chlorination takes place) can also act as oxidisers. It is also assumed that some bacteria can accelerate the corrosion of ductile iron. This all results in the formation of iron deposits on the pipe wall, causing an increased hydraulic resistance and adverse effect on water quality. By the release of iron, the water turbidity increases and colour appears (yellow-brown, or green in anaerobic environments). Lead absorbs, or is built into iron particles, which results in a higher lead intake through drinking water. The following water composition can be recommended:
$pH > 7.5$
$TIC > 0.5$ mmol/l
$((Cl^-) + 2(SO_4^{2-}))/TIC < 1$

Copper: Two forms of corrosion are typical for copper: uniform corrosion and pitting. Uniform corrosion starts with a chemical reaction between copper and dissolved oxygen (other components such as free chlorine can sometimes also act as oxidisers) and results in the formation of a layer of insoluble copper salts. These layers, usually a green colour, normally stabilise after one or two years without serious effects on the pipe material. However, taste and discolouration may appear due to the copper release.

Pitting corrosion is a more serious problem that can occur only a few months after the installation of the pipe. It appears as a consequence of either poor material quality (carbon particles left behind on the pipe wall during manufacturing), the installation technique (aggressive fluxes and residuals produced during soldering) or water stagnation (particles precipitation). The risk of leakage caused by pitting corrosion can be diminished by increasing the pH, bicarbonate concentration and organic matter. Oxygen levels and particle concentration should decrease. The recommended values are:
$TIC > 2$ mmol/l, to prevent pitting corrosion and
$pH > 0.38TIC + 1.5(SO_4^{2-}) + 5.3$, to prevent copper release.

Dezincification

Brass: Zinc present in this alloy dissolves in contact with water (*dezincification*). The location affected by this process becomes porous in time resulting in fractures and leaks. The zinc release in the water is insignificant. Experiments have shown that chloride ions affect the rate

of dezincification while bicarbonate ions determine the occurrence of the process. High pH values promote formation of the deposits on the pipe wall that can cause clogging. The recommendations are:
$pH < 8.3$
$TIC > 4$ mmol/l
$(Cl^-) < 40$ mg/l

Finally, the authors of the study give the following general recommendations for optimal water composition in a distribution system:
7.8 and $(0.38TIC + 1.5(SO_4^{2-}) + 5.3) < pH < 8.3$
$TIC > 2$ mmol/l
$-0.2 < SI < 0.3$
$((Cl^-) + 2(SO_4^{2-}))/TIC < 1$

The basis for the upper limit of the saturation index (0.3) is the need to inhibit scaling in hot water apparatus.

Corrosion and soil aggressiveness
Corrosion may also result from soil aggressiveness. The main factors that influence the aggressiveness are listed in Table 6.7 (van der Zwan and Blokland, 1989). By evaluating each factor as proposed in the table, the soil is considered to be:
– not aggressive if the sum of points is > 0,
– slightly aggressive if the sum is between -4 and 0,
– aggressive if the sum is between -10 and -5,
– very aggressive if the sum is < -10.

Corrosion control
The ways of achieving corrosion control are:
1 selection of adequate materials and design concepts,
2 modification of water quality parameters,
3 use of inhibitors,
4 cathodic protection,
5 use of corrosion-resistant linings, coatings and paints.

Material selection: A proper material choice can reduce corrosion, though it is just one of the aspects to be taken into account. Several alternative materials are usually compared based on the cost, availability, ease of installation and maintenance, as well as resistance to corrosion. Compatible materials should be used throughout the system as much as possible. Where this is difficult, galvanic corrosion can be avoided by placing insulating couplings made of different materials between the pipes.

Network design: Bad design of the pipes and structures may cause severe corrosion even in materials that may be highly resistant. Some of the important design considerations include:
– avoiding dead ends and stagnant areas,
– provision of adequate drainage where needed,

Table 6.7. Assessment of soil aggressiveness (van der Zwan and Blokland, 1989).

Parameters	Points	Parameters	Point
Soil composition		pH Reading	
Chalky soil, sand	2	pH > 6	0
Chalky clay, Marly chalk	1	pH ≤ 6	−2
Marly sand or clay, silty chalk	1	Potential (at pH = 7)	
Silt, silty sand, sandy clay, 75% mud	0	*E* > 400 mV	2
Peaty silt, silty marl	−1	*E* = 200–400 mV	0
Clay, Marly clay, humus	−2	*E* = 0–200 mV	−2
Marl, thick silt	−3	*E* < 0 mV	−4
Muddy & swampy ground, peat	−4	Carbonate concentration	
Soil condition		> 5%	2
Groundwater – none	0	1–5%	1
Groundwater – permanent	−1	< 1%	0
Groundwater – temporarily present	−2	H_2S and Sulphide	
Natural soil	0	None	0
Backfill	−2	Traces	−2
Same soil as trench	0	Present	−4
Different soil to trench	−3	Coal & coke	
Specific resistance		None	0
> 10000 ohms/cm	0	Present	−4
5000–10000 ohms/cm	−1	Chlorides	
2300–5000 ohms/cm	−2	< 100 mg/kg	0
1000–2300 ohms/cm	−3	≥ 100 mg/kg	−1
< 1000 ohms/cm	−4	Sulphates	
Water content		< 200 mg/kg	0
< 20%	0	200–500 mg/kg	−1
> 20%	1	500–1000 mg/kg	−2
		> 1000 mg/kg	−3

– selection of an appropriate flow velocity,
– selection of an appropriate metal thickness,
– reduction of mechanical stresses,
– avoiding uneven heat distribution,
– avoiding sharp bends and elbows,
– provision of adequate insulation,
– the elimination of grounding electrical circuits in the system,
– providing easy access to the structure for periodic inspection, maintenance and replacement of damaged parts.

Water quality adjustment: This is the easiest and most practical way to make water non-corrosive. However, it is not always effective bearing in mind possible differences in water quality at the sources. Two basic methods are: pH correction and oxygen reduction.

Most of the corrosion develops at low pH values. Chemicals commonly used for pH adjustment are:

1 lime, as $Ca(OH)_2$,
2 caustic soda, $NaOH$,

3 soda ash, Na_2CO_3,
4 sodium bicarbonate, $NaHCO_3$.

Oxygen is an important corrosive agent because it can act as an electron acceptor; it reacts with hydrogen and also with iron ions. These are all processes that promote corrosion, thus the reduction of oxygen can diminish their effects. Oxygen removal is rather expensive but some control measures can be introduced through the optimisation of aeration processes and the sizing of groundwater well and pumps that will avoid air entrainment.

Inhibitors

Inhibitors: Chemicals added to the water that form a protective film on the pipe surface are called inhibitors. This film provides a barrier between the water and the pipe, which reduces corrosion. Various products are used for this purpose, which can be classified in three main groups:
1 chemicals which cause $CaCO_3$ formation,
2 inorganic phosphates,
3 sodium silicate.

The difficulties in using inhibitors lay in the control of the process. Interrupted supply of the chemical can cause dissolution of the film and too low a dosage results in a fragmentary film, both of which increase pitting. On the other hand, excessive use of some alkaline inhibitors can cause an undesirable build-up of scale, particularly in harder waters. Finally, the flow rates must be sufficient to transport the inhibitor to all parts of the pipe surface; otherwise an effective film will not be formed.

Cathodic protection

Cathodic protection: Cathodic protection is an electrical method for preventing metallic corrosion. It forces the protected metal to behave as a cathode and therefore unable to release electrons. Basic methods of applying cathodic protection are:
1 the use of inert electrodes (high silicon cast iron or graphite) powered by an external source which forces them to act as anodes,
2 the use of magnesium or zinc as anodes that produces a galvanic reaction with the pipe material. Being more reactive than iron, they corrode, thereby keeping the pipe protected (*sacrificial corrosion*).

Sacrificial corrosion

Cathodic protection is expensive; it has to be controlled and renewed after some time. Apart from this it has little application in localities where corrosion has already started (holes, crevices, etc.).

Linings, coatings and paints: Corrosion can be kept away from the pipe wall if it is lined with a protective coating. The linings are usually applied mechanically, either during the manufacturing process or before pipe laying. They can also be applied to pipes in service, which is much more expensive. The most common pipe linings are:

1 epoxy paints,
2 cement mortar,
3 polyethylene.

Epoxy paints are used for steel and DI pipes. These are smooth coatings that are without any effect on the water quality, but are rather expensive and less resistant to abrasion (service life < 15 years).

Cement mortar is a standard lining for DI pipes, sometimes used for steel and cast iron. It is relatively cheap and easy in application. The disadvantage is the rather thick coating required which reduces the carrying capacity of the pipe. The rigidity of the lining may lead to cracking or sloughing.

Polyethylene coating is used for DI and steel pipes. This is a durable material ($\approx$50 years) with excellent characteristics (smooth and resistant) but also expensive.

6.2 NETWORK MAINTENANCE

Reactive and preventive maintenance

Operation and maintenance are often interrelated. Good operating systems that meet the consumer's requirements are also likely to reduce the level of maintenance. The maintenance considered in this context is predominantly consequential, or so-called *reactive maintenance* (e.g. repair of pipe bursts).

On the other hand, proper maintenance also contributes to the optimal operation of the system. As such, it is more of a condition, or requirement, for good operation, and is therefore understood as *preventive maintenance*.

Just as with the operation, efficient maintenance relies largely on good monitoring of the system. This is illustrated by the following example (Vreeburg *et al*., 1994). Figures 6.29 and 6.30 show the monitoring of turbidity in one CI pipe. In the first case, the higher turbidity is registered overnight, during longer retention times (low consumption), which indicates the release of corrosion products as a source of the problem. A possible remedy in this case is the control of the retention times through modification of the operation. When this problem occurs on a large scale, relining or pipe replacement should be carried out.

The next figure shows the turbidity in larger amplitudes, and higher during the day. The reason here is entirely different: large velocity (high consumption), which is causing re-suspension of the sediments in the pipe. The maintenance action taken in this case should be flushing of the pipe and coating renewal, if necessary.

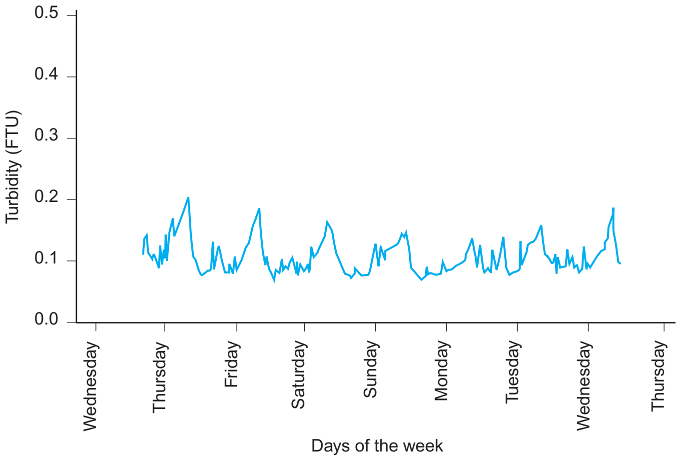


Figure 6.29. Turbidity during night flow (Vreeburg *et al.*, 1994).

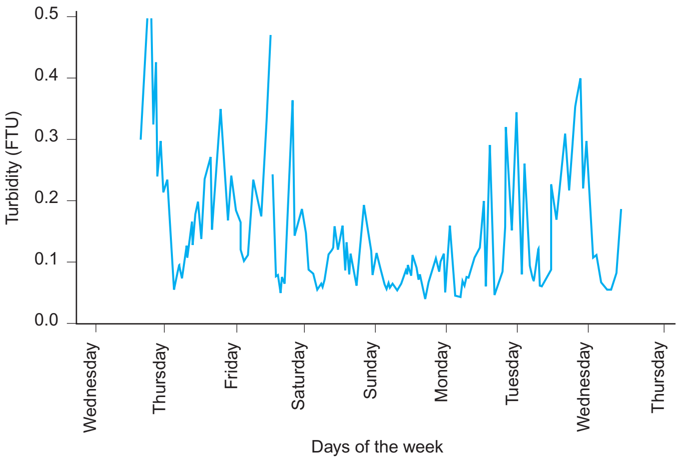


Figure 6.30. Turbidity during day flow (Vreeburg *et al.*, 1994).

6.2.1 *Planning of maintenance*

The selection of the type and level of maintenance follows a strategy based on the following principles:

– standard of service to the consumer should be regarded as a primary objective,
– within constraints set by the standards of service, decisions should be made on economic grounds.

Problems caused by the network deterioration determine the form of strategy. Their thorough description and good understanding is a prerequisite. The strategy should contain all the steps necessary for proper economic decisions. It should be future oriented and therefore not only cope with current problems. It should also be sufficiently flexible to allow for easy incorporation of improvements in technology. Major stages of the strategy to be followed are listed in Figure 6.31.

Practical factors that influence the strategy have already been discussed in previous sections and are only summarised here:

1 design and technical layout,
2 soil conditions,
3 surface activities,
4 climate,
5 water quality,
6 material selection,
7 construction methods,
8 pressures and flows.

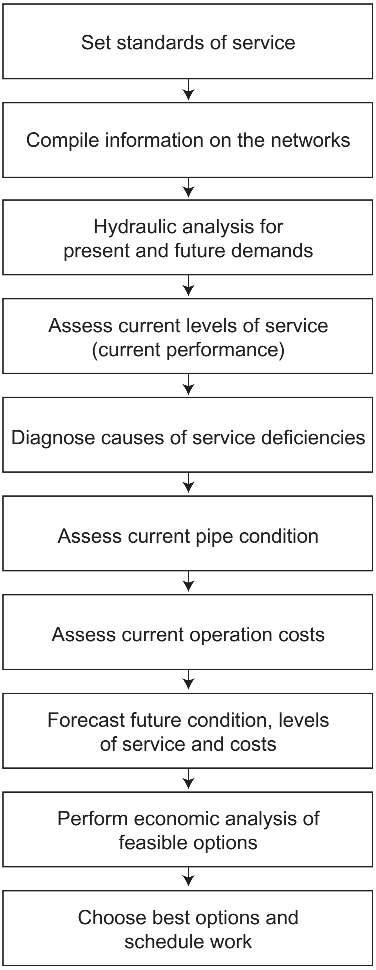


Figure 6.31. Organisation of maintenance (Brandon, 1984).

Preference for either preventive or reactive maintenance is derived from the strategy selected. Generally speaking, the annual costs of repairs and cleaning operations responding to the consumers' complaints are smaller than the annuities of main rehabilitation and replacement. However, this is only true for the standard frequency of pipe bursts. The expected trend is that the future number of ruptures will increase. Preventive maintenance can extend the economic lifetime of the system, and therefore is a 'must'; only the level of maintenance is debatable.

6.2.2 *Pipe cleaning*

When disturbed, corrosion deposits in pipes or sediments caused by improper treatment have to be removed in order to prevent water quality deterioration. The decision to conduct pipe cleaning results from the following situations:

– consumers' complaints about water quality (colour, turbidity),
– after a new pipe has been laid or an existing pipe has been repaired,
– the need for removal of excessive disinfectant used to kill bacteria or living organisms in pipes,
– systematic cleaning as a part of regular (preventive) network maintenance.

Three techniques of pipe cleaning are commonly used: flushing, air scouring and swabbing (or pigging).

Flushing
By opening a hydrant or washout on the main, an increased water flow is generated to remove loose deposits. Approximate velocities required for transport of sand particles ($\rho = 2650$ kg/m^3) according to experience in Great Britain are listed in Table 6.8.

Standard flushing velocities applied in The Netherlands are generally lower: around 1.5 m/s due to finer sediment and the use of corrosion free pipes in the system. As a general guideline, the approximate quantity of water needed is equivalent to three full volumes of the pipe that is being flushed.

Table 6.8. Required flushing velocity for sand particles, d = 0.2 mm (Brandon, 1984).

Pipe diameter (mm)	v (m/s)	Q (l/s)
50	1.3	2.7
75	1.6	7.2
100	1.8	15.0
150	2.2	41.0
200	2.6	83.0

Flushing is a simple method of cleaning, but not always efficient. The disadvantages are:
– large amounts of water used (particularly in large diameters),
– the velocity increase in the pipe being flushed may disturb the flow and pressure pattern upstream of the cleaned section,
– in areas with progressive corrosion, flushing offers only a partial improvement,
– not all parts of the distribution system may be equally suitable for the generation of high velocities (e.g. in low pressure areas).

Flushing pipes as a preventative measure requires good planning. The following factors are to be taken into consideration:
– selection of the optimal pipe route,
– the location of valves that are operated in order to isolate the flushed route from the rest of the system,
– total length of section that is flushed in one run,
– choice of hydrants (number and location) that will have to be opened in order to generate the necessary velocity,
– proper sequence of routes to be flushed.

The target of any flushing plan is to clean the system efficiently i.e. with the minimum quantity of water possible, as well as with the minimal operation of hydrants and valves. Particularly in looped systems, making an optimal flushing plan is almost impossible without the use of a network computer model.

The example in Figure 6.32 shows monitoring of the effects of flushing in a pipe section, which was conducted by the water company 'Europoort' in Rotterdam in 1997. The diagram with recorded pressures, velocities and turbidity shows the second and third phase of flushing,

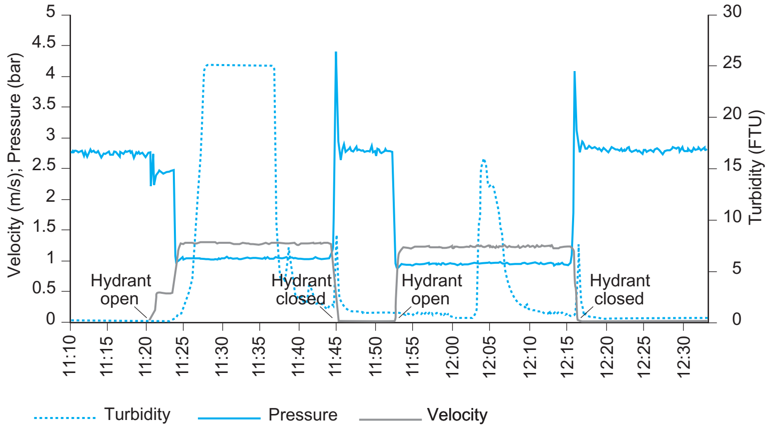


Figure 6.32. Example of a pipe flushing diagram.

each of approximately 20–25 minutes duration with a break of a few minutes in between. As a result of the hydrant opening, the normal service pressure of approximately 28 mwc (= 2.8 bar) drops to 10 mwc (= 1 bar) during flushing, while the flushing velocity will reach approximately 1.3 m/s. In this particular section, the initial turbidity of 64 ftu generated in the first phase was brought down to approximately 25 ftu in the second phase and eventually around 15 ftu in the third phase. The registered pressure surge of 40–45 mwc is a consequence of closing the hydrant, which should be conducted slowly.

Air scouring

In situations where water quantities available for pipe cleaning are limited, air scouring can be used as an alternative method to flushing. By this method, compressed air is injected into a continuous flow of water. Pushed by the air, the water will form into discrete slugs forced along the pipe at high velocities. The method is illustrated in Figure 6.33.

The atmospheric air will be injected in intervals of a couple of seconds and is filtered and cooled prior to the injection. A common injection point is at the hydrant but a service connection can also be used for this purpose. The length of the section that can effectively be cleaned is normally a few 100 metres.

The discharged water is left to spill around and will be collected by the sewer system. The jet leaving the hydrant is usually quite fierce. It is therefore important that the hose installed on the hydrant is securely fixed and ends with a protective plastic sheet or a box that dampens the vibrations (Figure 6.34).

Compared to ordinary flushing, air scouring requires both more equipment and energy supply. Control of the process may be a problem in some cases; the decision on optimal injection frequency is often empirical.

Swabbing

Swabbing is a cleaning technique by which the deposit is mechanically removed from the pipe. A cylindrical swab is inserted into a pipe and

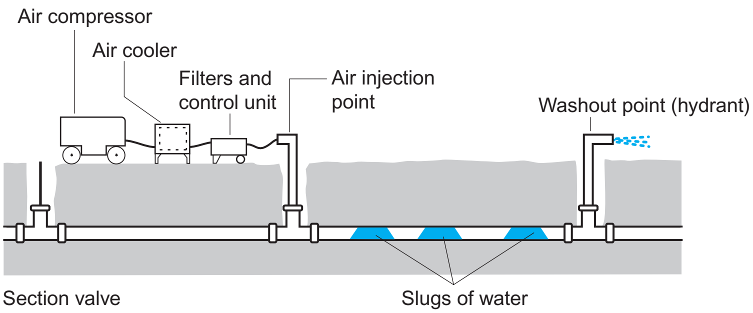


Figure 6.33. Air-scouring equipment and procedure.

Figure 6.34. Discharge point with dampening of the water jet.

driven along by the water pressure pushing the deposits ahead. The swab is porous and allows some water flow to pass, which enables the transport of deposits.

Swabs are usually made of polyurethane of different hardness. The hard grade is normally used for smaller reductions in pipe cross section (up to 30%). The soft grade is more convenient for larger reductions as it is compressible and can contract when it meets an obstruction in the main. When it is not possible to remove the obstruction, the swab is likely to be torn into small pieces, not further clogging the pipe.

Swabs made of abrasive materials are called pigs. They are stronger in construction and can be used for more aggressive cleaning (pigging). However, this should be avoided in the case of severe tuberculation as weak sections of the pipe could crack.

Different models of swabs and pigs are shown in Figure 6.35.

Swabs are normally oversized, compared to the corresponding pipe diameter (up to 25% larger). The process will be repeated with a number of units and visual inspection of the water at the end of the section, as well as the condition of the swab, will indicate the success of the cleaning process.

Pipe diameters up to ± 1000 mm can be treated with this method. Fire hydrants are the most common entry and exit points for small swabs while special fittings should be installed in the case of large diameters. The velocity of the swab will be similar to the normal operating velocities in the pipes.

Apart from being destroyed, swabs can be lost or jammed in the case of branches or connections that draw more water than the section that is cleaned. More expensive models contain a sensor so that their position in the pipe can be monitored. Nonetheless, proper care should be taken

Figure 6.35. Types of swabs and pigs (Manufacturer: Kleiss & Co.).

to isolate the cleaning section. A reversible flow may help in releasing the jammed swab.

6.2.3 *Animal disinfection*

Water quality in distribution pipes and reservoirs can be affected by the appearance of various types of organisms. Based on their preferred habitat, these can be classified in three groups:

1 those which swim freely; examples are Gammarus and Cyclops,
2 those that live in deposits, such as Chiranomid larvae,
3 those that are attached to the pipe surface; an example is Asellus, an organism that feeds on non-living organic matter.

The density and composition of animal populations in water distribution systems vary widely, and usually results from improper water treatment, insufficient disinfection, poor conditions of the pipes, insufficient hygienic precautions during the repair or replacement process or poor maintenance and lack of protection of the service reservoirs. For instance, Assellus typically occurs in the case of ineffective treatment of raw water containing algae, whilst Chiranomid larvae usually occur as a result of flying insects entering badly protected openings on the reservoirs. The problem is predominantly aesthetic and it is therefore a matter of maintaining animal numbers at levels 'invisible' to consumers. In some cases the aquatic organism can act as a host for parasites, such as Cyclops that can transmit the guinea worm, a dangerous parasite that appears in tropical and subtropical countries. The risk of infection is however low in piped systems that convey treated water. As most of the indications come from complaints, the assessment of animal presence through sampling is advisable as a preventive measure.

De-infestation can be done either by cleaning or by chemical treatment. Swimming and settled animals are relatively easily flushed out while those attached to the pipe have to be dislodged first, before being flushed. This can be done by swabbing or using air scouring technique.

Chemical treatment is carried out where flushing is insufficient. The chemicals commonly employed are chlorine, and occasionally pyrethrins and permethrin. When using chlorine, higher concentrations are required than normal dosages. Maintaining 0.5–1.0 mg/l of residual chlorine for a week or two will be sufficient in most cases (Brandon, 1984). In isolated and extreme cases, much higher dosages of 10–50 mg/l can be used, specifically for cleaning of the service reservoirs but these should be disconnected from operation. The concentrations applied during pre-chlorination may be effective in reducing the appearance of animals in the treatment works.

Pyrethrins and permethrin are types of pesticides that can be used effectively against Asellus, Gammarus and Chiranomid larvae. The recommended dose is 10 μg/l, which can be increased up to 20 μg/l, if necessary. The treatment should be done with the utmost precaution and in parts of the system fully isolated by valves. The normal contact time is 24 hours after which the pipes should be flushed with clean water; the recommended volume is twice the volume of the pipe (WHO, 2004). The flushed water should not be disposed of into natural streams as both of the substances are toxic to fish.

Long-term measures include the removal of organic matters (food restriction to the animals), which can be achieved by the following methods (Brandon, 1984):

1 improvement of the treatment process regarding suspended solids removal and animal penetration,
2 periodic cleaning of pipes and service reservoirs,
3 maintenance of a chlorine residual throughout the distribution system,
4 proper protection of openings on service reservoirs (i.e. ventilators with nets and manhole covers),
5 the elimination of dead ends and stagnant waters wherever possible.

6.2.4 *Pipe repairs*

Pipe repairs can be classified in two groups:

1 repairs of the damages caused by pipe transportation and handling at the site,
2 repairs of the damages in service.

The first group include correction of the pipe cross-section deformation (pipe re-rounding) and repairs of external and internal coatings; examples

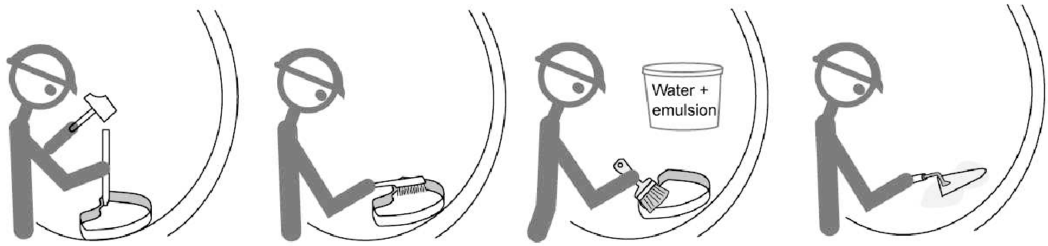


Figure 6.36. Repair of internal cement lining.

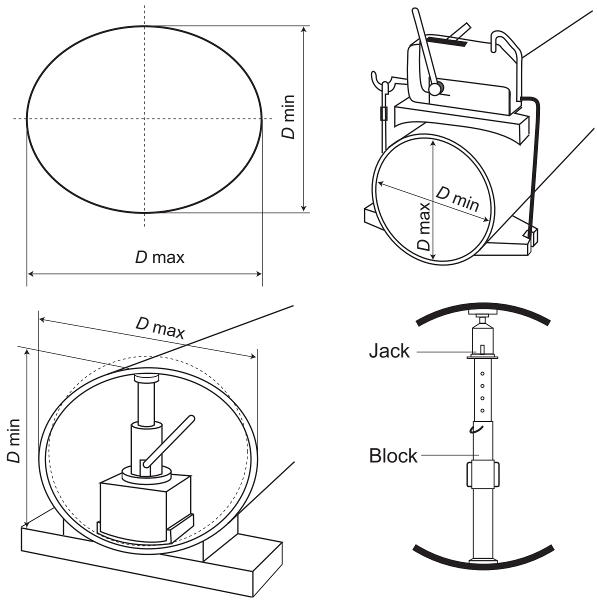


Figure 6.37. Correction of pipe deformation (Pont-a-Mousson, 1992).

for DI pipes are shown in Figures 6.36 and 6.37. Repairs to the pipes whilst in service can be executed at localities (circumferential failures and small holes) or along a section (longitudinal splits and blowouts). The second category is more complicated and sometimes requires replacement of the whole pipe. The use of *trench-less technology* may be considered here. By using appropriate equipment, the damaged pipe will be simply broken into pieces and the rubble pulled out leaving a tunnel for the new pipe. Owing to their strength and flexibility, PE pipes are often chosen in this situation.

Open-cut replacement follows the identical procedure as for the laying of new pipes. This is a more expensive alternative but offers a more flexible choice of material and size of the pipe. An example of open-cut section replacement is presented in Figure 6.38. The new piece of pipe

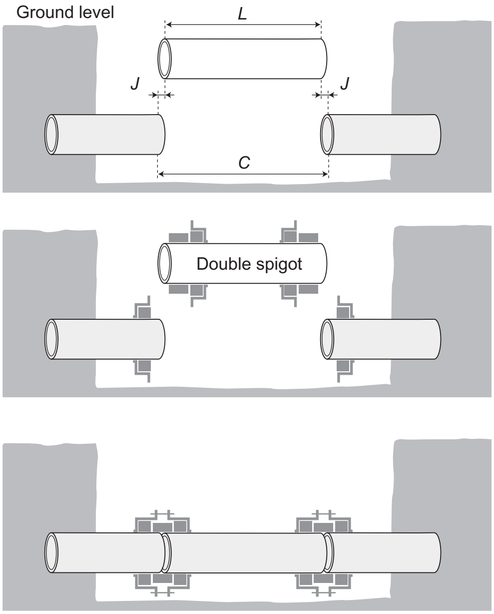


Figure 6.38. Pipe repair – section replacement.

Figure 6.39. Couplings used for connection of pipe pieces.

is cut at a shorter length ($L < C$) to allow a minimum space (J) for the coupling. Typical couplings used to connect pipes are shown in Figure 6.39.

Less damaged pipes can be repaired by relining. Conventional cement and epoxy are coatings exclusively used to prevent corrosion and

Figure 6.40. Structural relining of an old CI pipe.

have no structural function in the pipe. Cement relining is cheaper but less uniform in terms of results. The consequence is that the work on some sections may have to be repeated after the inspection has been carried out while at others, clogging of service connections might occur.

Real structural linings involve the installation of various types of hoses or pipes within the old pipe that will be tightly fitted to its wall. Such a solution improves the strength of the pipe to a large extent. Reduction in the original diameter, caused by the placing of an inner pipe, can be offset by the improved roughness so that this measure does not necessarily result in flow reduction (Figure 6.40).

6.3 ORGANISATION OF WATER COMPANY

The way in which operation and maintenance tasks are implemented has implications for the organisational set-up of the water company. The availability of technical means and expertise in itself will not be enough if good planning of activities and coordination between various services are missing.

The entire management of the water distribution is usually taken care of by a department that is part of a larger water supply company. Such a department has to function in line with the general policies of the company, providing well-distinguished tasks and responsibilities among the employees.

6.3.1 *Tasks*

The activities of the distribution department (company) are basically divided into office work and fieldwork, some of these being centralised.

Hence, the global structure consists of the head office and a number of district centres. The most important tasks of the central office are:
– collection of technical data (mapping),
– design of the main network,
– financial aspects of the network design and operation,
– monitoring of the network operation (control centre),
– control of major consumers.

More practical responsibilities of district centres are:
– construction of the network and service connections,
– preventive maintenance (repair and cleaning),
– failure service,
– installation and maintenance of water meters,
– leakage detection and repair,
– water quality control,
– control of indoor installations,
– connection and disconnection of the consumers,
– management of the stock of spare parts,
– measurements in the network,
– registration of technical data,
– administration of the activities.

Depending on the area supplied, some tasks can be reallocated between the district centre and the head office.

6.3.2 *Mapping*

Maps are the starting point of any maintenance. A detailed and regularly updated database is therefore of paramount importance. The scale and level of information will be dictated by the purpose of the map.

Maps used for design by means of computer modelling consist of distribution pipes usually with a diameter larger than 80 mm. The main information offered here concerns pipe routes, diameters and materials. Pipe junctions and crossings, as well as the position of other components (in particular, valves) have to be clearly indicated. A common map scale for this purpose is 1:1000–1:5000. Such maps are nowadays often digitised for computer use, providing quick access and easy correction and storage. An example is shown in Figure 6.41.

Maps for maintenance purposes are of a smaller scale and with more detailed information. These have to show the location of service pipes, valves, house connections, etc. enabling an efficient reaction when problems occur. If made by computer, the maps can be created with different sorts of information and overlaid, which is convenient for analyses (e.g. separate maps showing topography, houses and streets, pipe network, sewer network, electricity and gas, etc.). In other cases they are commonly drawn on transparent paper (Figure 6.42).

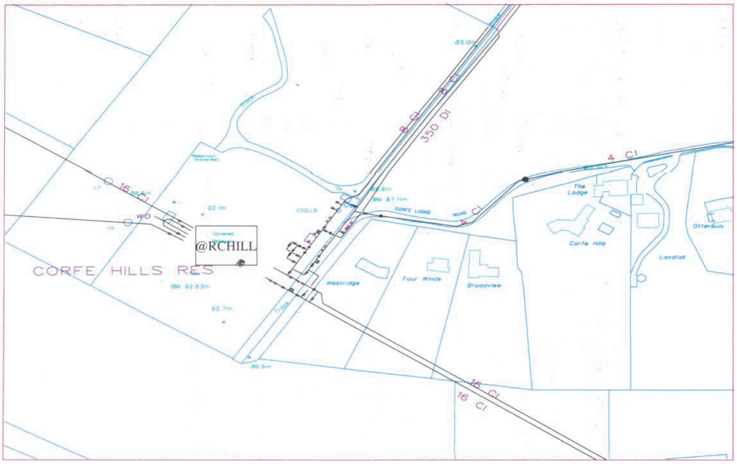


Figure 6.41. Computer-drawn map of a network section (Wessex Water PLC, 1993).

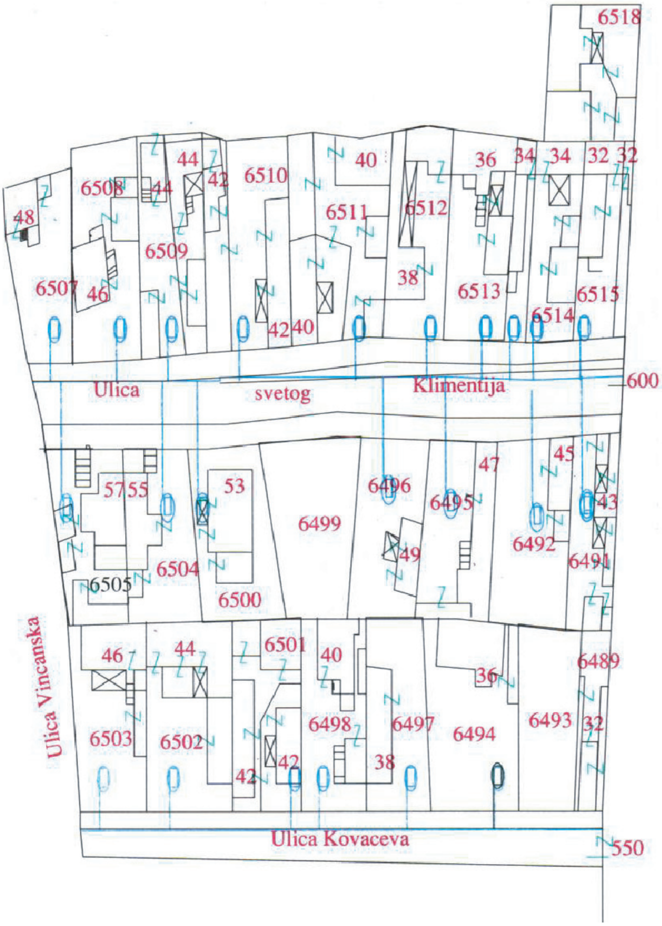


Figure 6.42. Map of residential area (Obradović, 1991).

The recent introduction of geographical information systems (GIS) into water distribution system management has greatly enhanced the amount and accuracy of data available. GIS maps have become readily available and can receive any additional information that becomes available after any replacement, connection or disconnection or expansion of the system has taken place. In this way, these maps enable multiple use: providing direct input for the computer model, accurate billing information and the location of system components that are malfunctioning and have to be repaired, etc.

An example of a GIS-based computer model is shown in Figure 6.43.

6.3.3 *Structure and size*

Construction

Construction work in the system can be carried out by the company or by a hired contractor. This decision determines the degree of investment required in manpower and mechanisation.

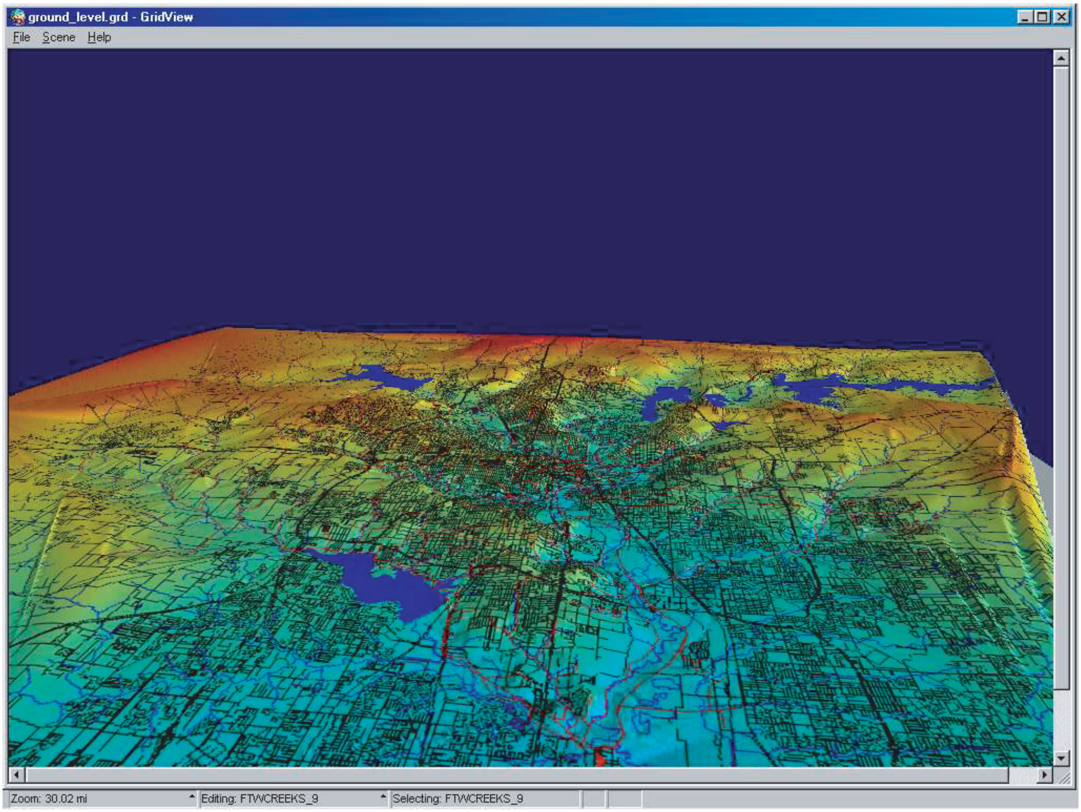


Figure 6.43. 3-D map of an urban distribution system (Wallingford Software Ltd).

Having work carried out by specialised companies has the following advantages:
– engaged labour and mechanisation is always appropriate to the amount of work,
– the work of the contractor can be partly carried out in conjunction with other services (gas, electricity, etc.) which reduces the costs for the water distribution company.

Despite the fact that labour costs in the developing world are relatively cheap, keeping an in-house construction section would appear to be fairly expensive, due to the inefficient employment of resources.

Maintenance
This is usually the largest section in the company. The number of employees is related to the size of the network but also to the required level of maintenance; loss of labour due to illness or holiday should also be taken into account.

Contractors can sometimes implement specific maintenance tasks, as well.

Failure service
The number of people required is dependant on the intended goals to be achieved. Where a 24-hour supply is an objective, shifts are necessary. Good, mobile equipment and quick access to all parts of the network are required and distances, topography, quality of the roads, etc. have to be taken into account (Figure 6.44).

Figure 6.44. Mobile workshop for quick repair service.

Investment in equipment, as well as proper planning of the activity, can reduce the number of personnel required.

Supervision
The quality of workmanship has to be permanently supervised. A small number of qualified staff is involved here, responsible also for formulation of conditions, such as:
– selection of the materials,
– standardisation of working procedures,
– standardisation of control procedures (testing of installations, conditions for connecting and disconnecting, etc.).

Supervision can sometimes be done externally in the district, by the head office.

Administration
This is a supporting element of the company. A good, not necessarily extensive, record about the management of the company must be kept. The number and profile of the people depends on the type of the activity. Where there is a head office with administrative staff, most of these tasks are done in that place. At least the technical records of all connections are kept in the district centre (for the failure service); together with administrative records about the employees, use of the materials, equipment, etc.

6.3.4 *Example*

An example of typical water distribution management in The Netherlands is illustrated in this paragraph.

Water Supply Company 'Drenthe' (WMD) was founded in 1937 with a small distribution system serving a few thousand people from less than one hundred connections in a rural area. Nowadays it is the biggest water company in the province. Basic information about the company is given in Table 6.9 (statistics in 2001).

Table 6.9. Water Supply Company 'Drenthe' – general data (2001).

Area covered	2200 km^2
Population served	450,000
Number of connections	180,000
Distribution of pipelines	4000 km
Service pipes	4800 km
Production	32,000,000 m^3
Average day	90,000 m^3
Maximum day	152,000 m^3
Number of employees	179
Predominate pipe materials	PVC-54%
	AC-38%

Organization setup

The company is non-profit based, owned by the municipalities supplied and the province (50–50). Representatives of each group, together with the company director, form the Board of Supervisors, which is the actual governing body. The structure of the company is shown in Table 6.10.

The organisation, headed by a director, consists of four departments: production, distribution, finance and personnel. The heads of these sections report to the director. Over 50% of employees work in the distribution department, which is divided into three districts. One of the district centres looks as follows (Table 6.11):

Annual investment in the district network is in the order of 700,000 US$ with about 800 new connections and 15 km of pipeline. The value of the stock is approximately 100,000 US$.

Activities in the design phase

Each project, after getting approval from the board of directors, goes to the planning stage. This mainly comprises of the following data collection and necessary investigations:

Socio-economic survey: The data are available in the municipality and other corresponding institutions.

Table 6.10. Water Supply Company 'Drenthe' – structure of the company (2001).

Departments	Head-office	Districts	Total
Director	1	0	1
Production	19	8-workshop/18-PST	45
Distribution	2	94	96
Finance	13	0	13
Personnel	24	0	24
Total	59	120	179

Table 6.11. Water Supply Company 'Drenthe' – sample district.

Covered area	700 km^2
Served population	150,000
Number of connections	60,000
Distribution pipelines	1300 km
Production in 2001	12,000,000 m^3
Number of staff	27
Manager/deputy manager	1/1
Supervisors	3
Inspector	1
Fitters	18
Storekeeper	1
Administrator	2

Topographic survey: Supplied from other sources.

Soil investigation: No regular soil investigation is carried out; only special cases can be considered (e.g. road crossings).

Existing services: A good coordination is provided among other services in the area. In the case of new extensions, it is common practice to lay all service lines in the same trench and at the same time. Maps with precise routes of all services are available for any interested organisation or individual.

Design: In most cases the design is carried out within the distribution section of the company by experienced technical staff with sufficient working facilities (a senior design engineer and a number of assistants). About 80 km of pipelines is designed per year, on average. All the data and originals of the computerised drawings are preserved in the head office. The district offices can access the database at any time. The distribution section does service connection designs as well.

Bearing in mind the soil conditions, PVC is the most preferred material for up to 500 mm diameter. The minimum design pressure is 25 mwc above street level. Leakage of 10% is taken into account. The demand variations are represented by the peak factor 1.50 for the maximum consumption day, and 1.68 for the maximum hour consumption. Other common design standards are:

– house connection, $D = 25$ mm
– < 20 connections, $D = 80$ mm
– > 20 connections, $D = 100$ mm
– design velocities, $v = 0.5$–1.5 m/s
– fire hydrants, each 100–200 m.

Construction works
In 1955 the company covered about 26,000 connections and employed 120 workers (fitters and diggers), plus approximately 200 on a temporary basis. At present, this number is drastically reduced. Smaller construction works are carried out within the company by the maintenance section. For larger activities a contractor is engaged.

Stock of spare parts: Each district has its own storage for pipes, fittings and other equipment and materials. Transport to the working place is the responsibility of the contractor.

Quality control: All components built in the system carry the KIWA certificate. Any damage to the pipes caused during the manufacturing process is repaired at the manufacturer's cost, no matter when the pipe was produced.

Pipe laying: The soil is usually sandy, therefore it is not usual to place extra bedding under the pipe. PVC distribution pipes are joint by dual socket 'push-in' joints. Each pipe is cleaned before laying. The backfilling is compacted by mechanical vibrators. The pipeline is usually tested after the backfilling, at a pressure of 90 mwc, for two hours. Pipelines are disinfected with chlorine before commissioning.

Work supervision: One field engineer is responsible for the full-time supervision of the pipe laying. He is provided by a car and wireless communication system. The contractor is totally responsible for the quality of the work. Supervision for smaller works is carried out part time.

Service connections: In most cases the connections are made by the company's fitters. For new residential areas with many service lines this is done by the contractor. All components for the service connections are supplied from the stocks of the company (the KIWA certificate is required here as well). One field technician is responsible for the work supervision.

Operation and maintenance
As-built drawings: This information is computer processed. The detailed position and description of the valves, fittings, washouts, pits, crossings, etc. is indicated on the map layout, sufficient for all information necessary for the maintenance activities.

Pipe flushing: This is done at two-yearly intervals. The water for this purpose is drawn from the system.

Pipe replacement: The pipe is normally replaced in three instances:
1 when there is frequent leakage at the same segment,
2 when the route has to be diverted,
3 due to increase in the capacity.

System monitoring: In selected points in the system (mostly the ends of the system), the pressure is monitored continuously but only during the seasonal peaks in summer. Pressures and flows are measured automatically in all pumping facilities, during the whole year. All records are preserved in the head office.

Leakage: The leakage level in the system is about 5% of total production. It is predominantly (by number of leaks) in the service lines. Most of the breakages in the distribution system are registered on AC pipes. No leakage detection programme exists due to the low leakage rate. Very often the consumers report the leaks. Precise evidence about the breaks is recorded in the computer. Leak repairs normally take 3–4 hours and this service is available 24-hours a day. The team on duty has a vehicle equipped with all the necessary tools. Outside regular working hours the vehicle is always with one of the workers, and when required they can drive directly to the place of failure, or to the district centre if additional information is necessary.

Metering: All service connections in the system have water meters installed. The reading of domestic consumers' meters is carried out once a year and for large consumers four times a year. The meters are replaced every 7–10 years. The company has its own workshop for the maintenance of the meters. On average about 20,000 meters are replaced or rehabilitated per year. All records with respect to meters are computerized. The estimated meter under-registration is approximately 1.5% of the total delivery into the system.

Training and research: There are 'on the job' and 'off the job' training programmes. A compulsory two-year training for the technical staff is organised by VEWIN. The company arranges different types of short training programmes. The company does not invest substantially in research and development, which is carried out in cooperation with the KIWA institute.

APPENDIX 1

Workshop Problems

A1.1 WATER DEMAND

PROBLEM A1.1.1
Determine the production capacity of a treatment installation for a city with a population of 1,250,000. Assume a specific consumption per capita of 150 l/d, non-domestic water use of 30,000,000 m^3/y and UFW of 12%.

Answer:
Q_{avg} = 112 million m^3/y or 3.6 m^3/s

PROBLEM A1.1.2
A water supply company delivers an annual quantity of 15,000,000 m^3 to a distribution area of 100,000 consumers. At the same time, the collected revenue is 6,000,000 US$, at an average water tariff of 0.5 US$/$m^3$. Determine:

a the delivery on an average consumption day,
b the percentage of unaccounted-for water,
c the specific consumption per capita per day, assuming 60% of the total delivery is for domestic use.

Note:
b Express the unaccounted-for water as a percentage of the delivered water.

Answers:
a Q_{avg} = 41,096 m^3/d or 1712 m^3/h
b UFW = 20%
c q = 247 l/c/d

PROBLEM A1.1.3
A family of four pays for annual water consumption of 185 m^3. Determine:

a the specific consumption per capita per day,
b the instantaneous peak factor at a flow of 300 l/h.

Answers:
a q = 127 l/c/d
b pf_{ins} = 14

PROBLEM A1.1.4

An apartment building of 76 occupants pays for an annual water consumption of 4770 m^3. Determine:

a the specific consumption per capita per day,
b the instantaneous peak factor during the maximum consumption flow of 5.5 m^3/h.

Answers:

a q = 172 l/c/d
b pf_{ins} = 10

PROBLEM A1.1.5

A residential area of 1200 inhabitants is supplied with an annual water quantity of 63,800 m^3, which includes leakage estimated at 10% of the total supply. During the same period, the maximum flow registered by the district flow meter is 25.4 m^3/h. Determine:

a the specific consumption per capita per day,
b the maximum instantaneous peak factor.

Note:

a Specific consumption should not include leakage.
b Peak factors include leakage unless the flow is measured at the service connection.

Answers:

a q = 131 l/c/d
b pf_{ins} = 3.5

PROBLEM A1.1.6

A water supply company delivers an annual volume of 13,350,000 m^3. The maximum daily demand of 42,420 m^3 was observed on 26 July. The minimum, observed on 30 January, was 27,360 m^3. The following delivery was registered on 11 March:

Hour	1	2	3	4	5	6	7	8	9	10	11	12
m^3	433	562	644	835	1450	1644	1856	1922	1936	1887	1721	1712
Hour	13	14	15	16	17	18	19	20	21	22	23	24
m^3	1634	1656	1789	1925	2087	2055	1944	1453	1218	813	676	602

Determine:

a delivery on an average consumption day and the range of seasonal peak factors,
b the diurnal peak factor diagram,
c the expected annual range of peak flows supplied to the area.

Answers:

a Q_{avg} = 36,575 m^3/d; pf_{sea} = 0.75–1.16
b Q_{avg} = 1435.6 m^3/h

Hour	1	2	3	4	5	6	7	8	9	10	11	12
pf_h	0.302	0.391	0.449	0.582	1.010	1.145	1.293	1.339	1.349	1.314	1.199	1.193
Hour	13	14	15	16	17	18	19	20	21	22	23	24
pf_h	1.138	1.154	1.246	1.341	1.454	1.431	1.354	1.012	0.848	0.566	0.471	0.419

Note that 11 March is not an average consumption day. The average flow derived from the annual quantity is $Q_{avg} = 1524$ m^3/h.

c $Q_{max} = 2563$ m^3/h; $Q_{min} = 343$ m^3/h

PROBLEM A1.1.7

Estimated leakage in the area from Problem 1.6 is 20% of the daily supply. The leakage level is assumed to be constant over 24 hours. Calculate the hourly peak factors for the actual consumption on 11 March.

Note:

Leakage of 20% means a constant flow (loss) of 287.1 m^3/h.

Answer:

$Q_{avg} = 1148.5$ m^3/h

Hour	1	2	3	4	5	6	7	8	9	10	11	12
pf_h	0.127	0.239	0.311	0.477	1.013	1.181	1.366	1.424	1.436	1.393	1.249	1.241
Hour	13	14	15	16	17	18	19	20	21	22	23	24
pf_h	1.173	1.192	1.308	1.426	1.567	1.539	1.443	1.015	0.811	0.458	0.339	0.274

PROBLEM A1.1.8

The consumption calculated in Problem A1.1.7 consists of three categories: domestic, industrial and commercial. The industrial category contributes to the overall consumption with a constant flow of 300 m^3/h, between 8 a.m. and 8 p.m. The commercial category requires a flow of 100 m^3/h, between 8 a.m. and 4 p.m.

a Determine the hourly peak factors for the domestic consumption category.

b Assuming the industrial and commercial consumption to be constant throughout the whole year, calculate the average consumption per capita if there are 150,000 people in the area.

Answers:

a $Q_{avg} = 965.2$ m^3/h

Hour	1	2	3	4	5	6	7	8	9	10	11	12
pf_h	0.151	0.285	0.370	0.568	1.205	1.406	1.626	1.279	1.294	1.243	1.071	1.062
Hour	13	14	15	16	17	18	19	20	21	22	23	24
pf_h	0.981	1.004	1.142	1.386	1.554	1.521	1.406	1.208	0.965	0.545	0.403	0.326

b $q = 166$ l/c/d

PROBLEM A1.1.9
The registered annual domestic consumption is presently 38.2 million m^3. Determine:

a the consumption after the first 10 years, assuming an annual population growth of 3.8%,
b the consumption after the following 10 years (11–20) assuming an annual population growth of 2.2%.

Compare the results of the Linear and Exponential models discussed in Paragraph 2.4.

Answers:
a In 10 years from now: Q_{lin} = 52.7 million m^3; Q_{exp} = 55.5 million m^3
b In 20 years from now: Q_{lin} = 64.3 million m^3; Q_{exp} = 69.0 million m^3

PROBLEM A1.1.10
The following annual consumptions were registered in the period 1990–1995 (in million m^3):

Year	1990	1991	1992	1993	1994	1995
Q (10^6 m^3)	125.4	131.8	138.2	145.4	152.6	159.9

Make a forecast for the year 2005.

Answer:
Q_{2005} = 260.7 million m^3 (exponential growth of 5%)

A1.2 SINGLE PIPE CALCULATION

PROBLEM A1.2.1
A pipe of length L = 500 m, diameter D = 300 mm and absolute roughness k = 0.02 mm transports a flow Q = 456 m^3/h. Determine the hydraulic gradient by using the Darcy–Weisbach formula. The water temperature may be assumed to be 10°C. Check the result by using the hydraulic tables in Appendix 4.

Answer:
By using the Darcy–Weisbach formula, S = 0.0079.

From the tables for k = 0.01 mm, S = 0.007 if Q = 434.1 m^3/h. If S = 0.010, Q = 526.9 m^3/h. By linear interpolation: S = 0.0077, which is close to the calculated result.

PROBLEM A1.2.2
A pipe of length L = 275 m, diameter D = 150 mm and absolute roughness k = 0.1 mm transports a flow Q = 80 m^3/h. Determine the hydraulic gradient by using the Darcy–Weisbach formula. The water

temperature may be assumed to be 15°C. Check the result by using the hydraulic tables in Appendix 4.

Answer:
$S = 0.0108$;

From the tables for $k = 0.1$ mm, $S = 0.010$ if $Q = 76.7$ m^3/h.

PROBLEM A1.2.3
A pipe of length $L = 1000$ m and diameter $D = 800$ mm transports a flow $Q = 1.2$ m^3/s. Determine the hydraulic gradient:
a by using the Darcy–Weisbach formula for $k = 0.2$ mm,
b the Hazen–Williams formula for $C_{hw} = 130$,
c the Manning formula for $N = 0.010$ m$^{-1/3}$s.

The water temperature may be assumed to be 10°C.

Answers:
a $S = 0.0055$
b $S = 0.0054$
c $S = 0.0049$

PROBLEM A1.2.4
Determine the maximum capacity of a pipe where $D = 400$ mm and $k = 0.5$ mm at the maximum-allowed hydraulic gradient $S_{max} = 0.0025$. The water temperature equals 10°C. Check the result by using the hydraulic tables in Appendix 4.

Answer:
$Q_{max} = 429.8$ m^3/h

From the tables for $k = 0.5$ mm, $Q = 384.9$ m^3/h if $S = 0.002$ and 473.2 m^3/h for $S = 0.003$. By linear interpolation: $Q_{max} = 429.1$ m^3/h.

PROBLEM A1.2.5
Determine the maximum capacity of a pipe where $D = 200$ mm at the maximum-allowed hydraulic gradient $S_{max} = 0.005$:
a if $k = 0.01$ mm,
b if $k = 1$ mm.

The water temperature equals 10°C.

Answers:
a $Q_{max} = 123.1$ m^3/h
b $Q_{max} = 89.8$ m^3/h

PROBLEM A1.2.6
Determine the maximum capacity of a pipe where $D = 1200$ mm and $k = 0.05$ mm at the maximum-allowed hydraulic gradient:
a $S_{max} = 0.001$,
b $S_{max} = 0.005$.

The water temperature equals 10°C.

Answers:
a Q_{max} = 5669 m^3/h
b Q_{max} = 13,178 m^3/h

PROBLEM A1.2.7
Determine the maximum capacity of a pipe where D = 100 mm and k = 0.4 mm at the maximum-allowed hydraulic gradient S_{max} = 0.01. Use the Moody diagram. The water temperature equals 10° C.

Answer:
Q_{max} = 22.6 m^3/h

PROBLEM A1.2.8
Determine the pipe diameter that can transport flow Q = 720 m^3/h at the maximum-allowed hydraulic gradient S_{max} = 0.002. The pipe roughness k = 0.05 mm. Assume the water temperature to be 12°C. Check the result by using the hydraulic tables in Appendix 4.

Answer:
D = 477 mm; the first higher manufactured diameter D = 500 mm delivers 820.0 m^3/h.

From the tables for k = 0.05 mm and S = 0.002, Q = 818.2 m^3/h for D = 500 mm.

PROBLEM A1.2.9
A pipe, L = 450 m, D = 300 mm and k = 0.3 mm, conveys flow Q = 100 l/s. An increase in flow to 300 l/s is planned. Determine:
a the diameter of the pipe laid in parallel to the existing pipe,
b the pipe diameter if, instead of laying a second pipe, the existing pipe is replaced by a larger one,
c the pipe diameter if the existing pipe is replaced by two equal pipes.

For all new pipes, k = 0.01 mm. Assume the water temperature to be 10°C.

Note:
The present hydraulic gradient has to be maintained in all three options.

Answers:
For S = 0.007
a Q_2 = 200 l/s; D_2 = 363 mm (adopted D = 400 mm)
b Q = 300 l/s; D = 423 mm (adopted D = 500 mm)
c $Q_1 = Q_2$ = 150 l/s; $D_1 = D_2$ = 326 mm (adopted D = 350 mm)

PROBLEM A1.2.10
Find the equivalent diameters of two pipes connected in parallel, where L = 850 m and k = 0.05 mm, in the following cases:
a $D_1 = D_2$ = 200 mm; $Q_1 = Q_2$ = 20 l/s,

b $D_1 = D_2 = 400$ mm; $Q_1 = Q_2 = 100$ l/s,
c $D_1 = D_2 = 800$ mm; $Q_1 = Q_2 = 800$ l/s.

The water temperature equals 10°C.

Answer:
For $Q = Q_1 + Q_2$
a $S = 0.0020$; $D = 259$ mm (adopted $D = 300$ mm)
b $S = 0.0013$; $D = 520$ mm (adopted $D = 600$ mm)
c $S = 0.0021$; $D = 1042$ mm (adopted $D = 1100$ mm)

PROBLEM A1.2.11
Find the equivalent diameters of two pipes connected in series, where $L_1 = 460$ m, $L_2 = 240$ m, in the following cases:
a $D_1 = 400$ mm, $D_2 = 200$ mm; $Q = 80$ l/s,
b $D_1 = 200$ mm, $D_2 = 400$ mm; $Q = 80$ l/s,
c $D_1 = 600$ mm, $D_2 = 300$ mm; $Q = 400$ l/s.

Assume for all pipes that $k = 0.01$ mm and the water temperature is 10°C.

Answer:
For $L = 700$ m
a $S = 0.0087$; $D = 246$ mm (adopted $D = 250$ mm)
b $S = 0.0159$; $D = 217$ mm (adopted $D = 250$ mm)
c $S = 0.0239$; $D = 368$ mm (adopted $D = 400$ mm)

A1.3 BRANCHED SYSTEMS

PROBLEM A1.3.1
For the branched system shown in Figure A1.1, calculate the pipe flows and nodal pressures for a surface level (msl) in the reservoir that can maintain a minimum network pressure of 20 mwc. Assume for all pipes that $k = 1$ mm and the water temperature is 10°C.

Node	1	2	3	4	5	6	7	8	9	10
Z (msl)	—	18.2	26.5	16.2	13.6	16.3	14.8	13.1	11.3	12.8
Q (l/s)	−79.0	4.5	12.4	11.4	9.9	5.2	11.1	3.3	10.4	10.8

Answer:
The surface elevation of 52.5 msl at node 1 results in the pressures as shown in Figure A1.2. The minimum pressure appears to be in node 3 (20.3 mwc).

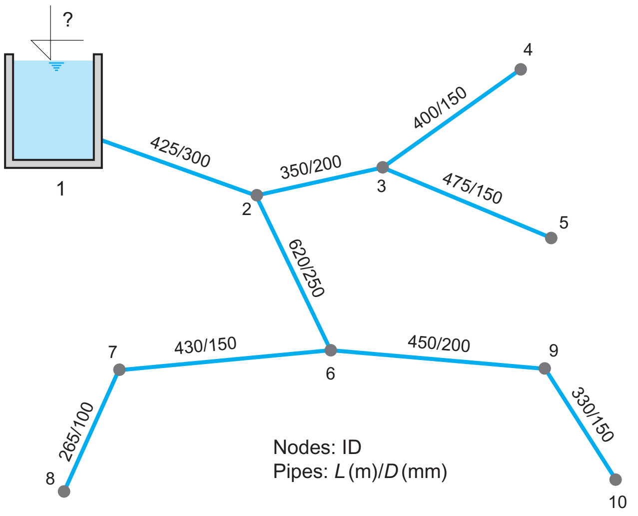


Figure A1.1. Network layout – Problem A1.3.1.

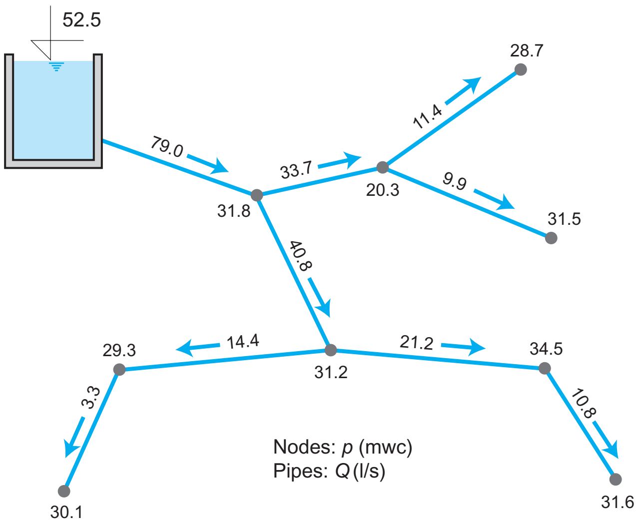


Figure A1.2. Pipe flows and nodal pressures – Problem A1.3.1.

PROBLEM A1.3.2

The minimum pressure criterion for the branched system shown in Figure A1.3 is 25 mwc. Determine the surface level of the reservoir in node 1 that can supply a flow of 50 l/s. What will be the water level in the second tank in this scenario? Calculate the pressures and flows in the system. Assume for all pipes that $k = 0.5$ mm and the water temperature is 10° C.

Node	1	2	3	4	5	6	7	8	9	10
Z (msl)	—	18.2	26.5	16.2	—	16.3	14.8	13.1	11.3	12.8
Q (l/s)	−50.0	7.6	16.4	9.2	−34.9	15.2	11.1	9.3	8.3	7.8

Answer:
See Figure A1.4.

PROBLEM A1.3.3
For the same system as in Problem A1.3.2 and the same surface levels in the reservoirs as shown in Figure A1.4, determine the pressures and

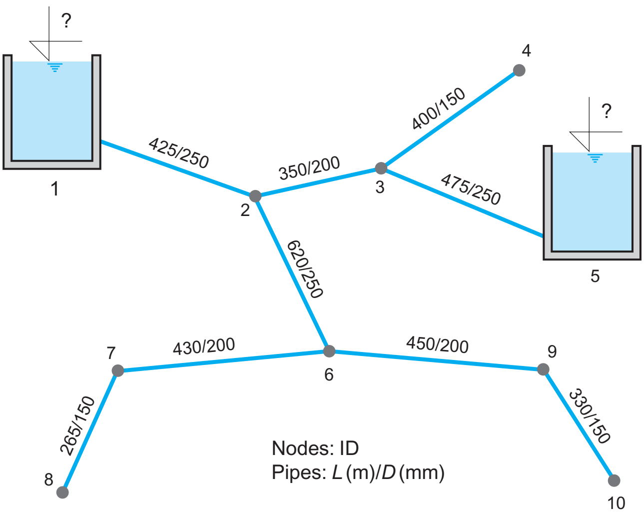


Figure A1.3. Network layout – Problem A1.3.2.

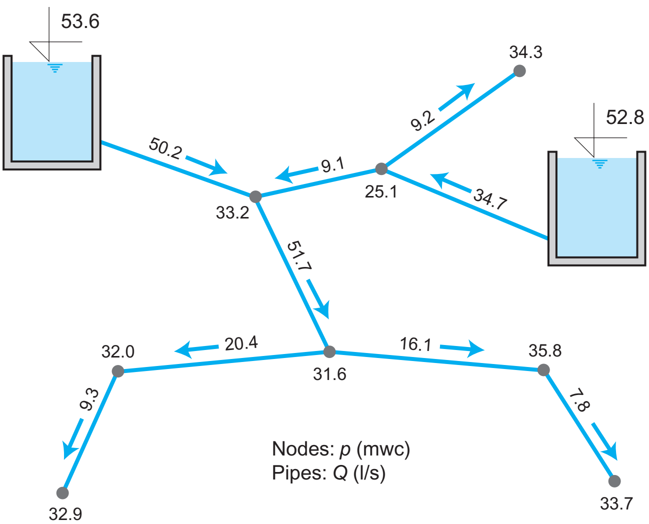


Figure A1.4. Pipe flows and nodal pressures – Problem A1.3.2.

flows if the demand in node 8 has increased for 10 l/s and in node 10 for 20 l/s.

Answer:
See Figure A1.5.

Due to the increase in demand, the minimum pressure point has moved from node 3 to node 10.

PROBLEM A1.3.4
Determine the pipe diameters for the layout shown in Figure A1.6, if the maximum-allowed hydraulic gradient $S_{max} = 0.005$. Determine the

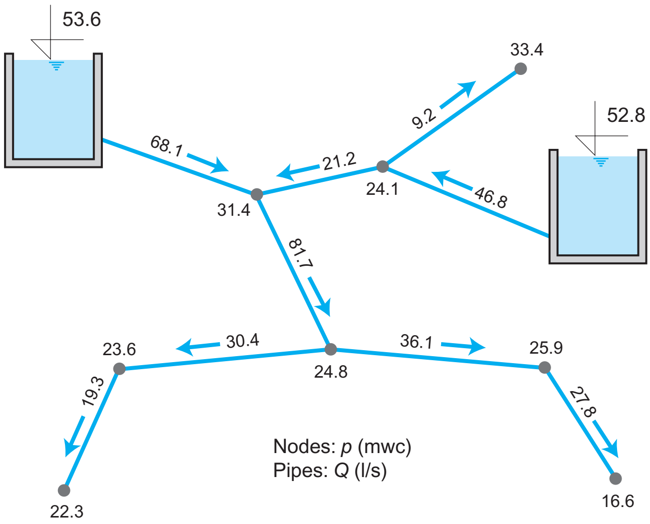


Figure A1.5. Pipe flows and nodal pressures – Problem A1.3.3.

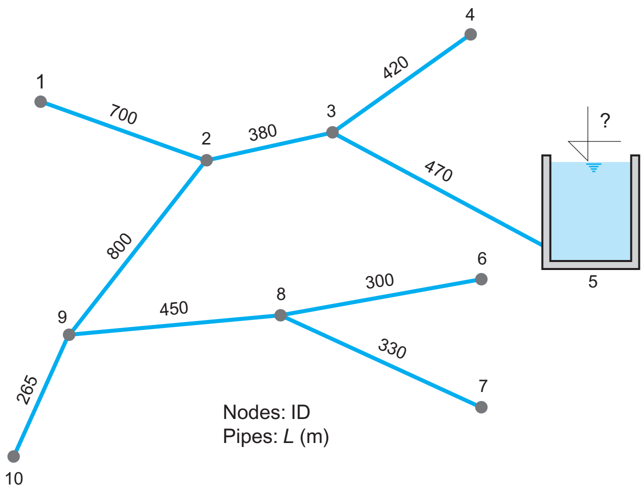


Figure A1.6. Network layout – Problem A1.3.4.

surface level of the reservoir at the supply point, which can maintain a minimum pressure of 20 mwc. Assume for all pipes that $k = 0.05$ mm and the water temperature is 10°C.

Node	1	2	3	4	5	6	7	8	9	10
Z (msl)	17.6	18.2	16.0	21.4	—	18.0	16.5	19.0	20.4	22.7
Q (l/s)	5.0	7.6	9.9	3.5	−58.1	3.8	4.4	10.5	9.2	4.2

Answer:
See Figure A1.7.

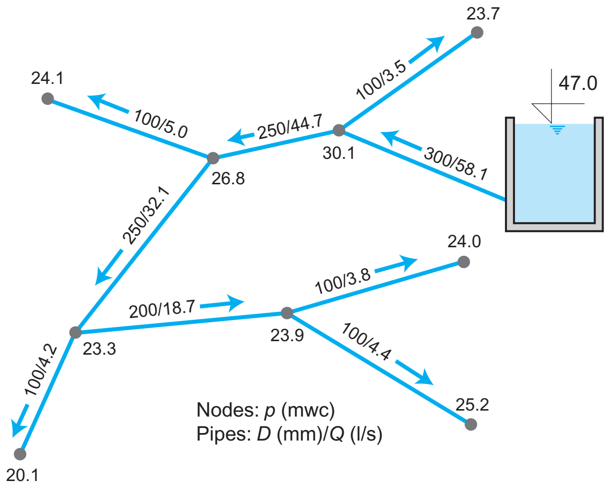


Figure A1.7. Pipe diameters/ flows and nodal pressures – Problem A1.3.4.

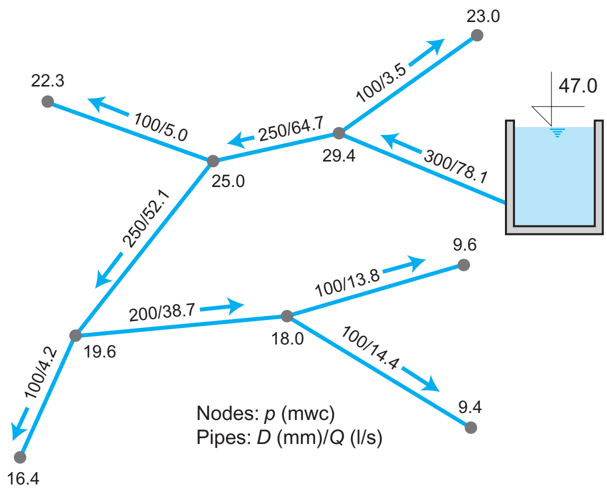


Figure A1.8. Pipe diameters/ flows and nodal pressures – Problem A1.3.5.

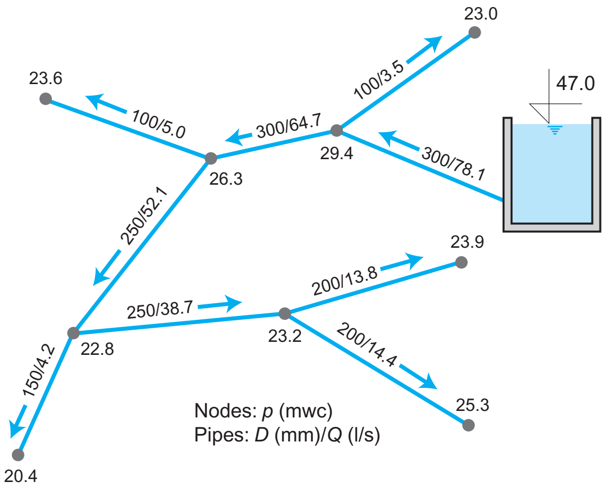


Figure A1.9. Pipe diameters/flows and nodal pressures – Problem A1.3.5.

PROBLEM A1.3.5

For the same system as in Problem A1.3.4 and the same surface level in the reservoir as shown in Figure A1.7, determine the pressures and flows if the demand in nodes 6 and 7 has increased for 10 l/s. Change the pipe diameters where necessary in order to meet the design criteria (S_{max} and p_{min}).

Answer:

By increasing the demand in nodes 6 and 7 to 13.8 and 14.4 l/s respectively, the pressures in the network will be as shown in Figure A1.8. Nodes 6 to 10 have pressure below 20 mwc. To satisfy the design pressure and hydraulic gradient, pipes 3-2, 2-9, 9-10, 9-8, 8-6 and 8-7 have to be enlarged (see Figure A1.9).

A1.4 LOOPED SYSTEMS

PROBLEM A1.4.1

For the same system as in Problem A1.3.3 and the same surface levels in the reservoirs as shown in Figure A1.5, determine the pressures and flows if nodes 3 and 9 are connected with a pipe, where $L = 780$ m, $D = 200$ mm and $k = 0.05$ mm.

Note:

Remove the branches and add their demand to the nodes of the loop 2-3-9-6. A ‘dummy’ loop, 1-2-3-5, should be formed to determine the flows from the tanks. Fixed $\Delta H_{1-5} = 53.6 - 52.8 = 0.8$ mwc is kept while balancing the heads throughout the calculation (see Figure A1.10).

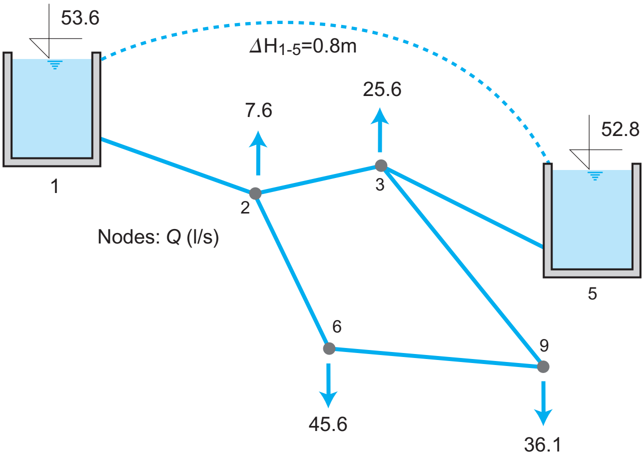


Figure A1.10. Network layout and nodal demands – Problem A1.4.1.

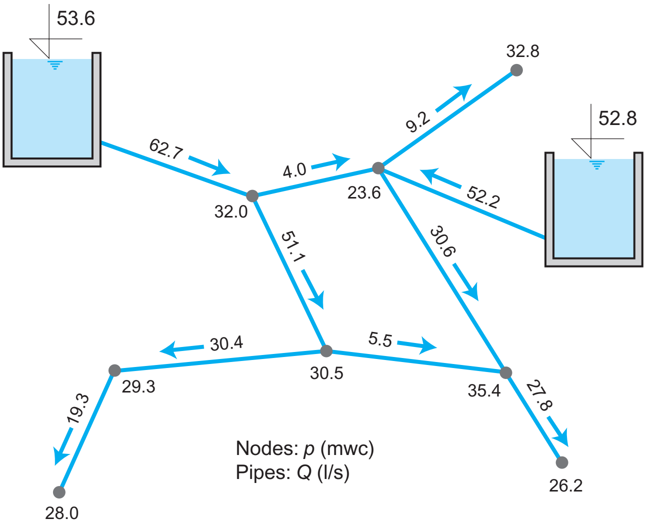


Figure A1.11. Pipe flows and nodal pressures – Problem A1.4.1.

Answer:
See Figure A1.11.

PROBLEM A1.4.2
For the same system as in Problem A1.3.5 and keeping the layout as shown in Figure A1.8, determine the pressures and flows if nodes 2 and 8 and 3 and 6 are connected with pipes with respective lengths of 680 and 470 m. For both pipes $D = 150$ mm and $k = 0.05$ mm.

Answer:
See Figure A1.12.

As the figure shows, pressures in the network will improve by creating loops.

PROBLEM A1.4.3

For the layout shown in Figure A1.12, analyse the pressure in the system:

a after the failure of pipe 9-8,

b after the failure of pipe 2-3.

What is the deficit of pressure to be provided at the supply point, in both cases?

Answers:

a There is no pressure deficit in the system caused by the failure of pipe 9-8 (Figure A1.13).

b In this case, Figure A1.14 shows a severe drop of pressure in the system. The observed deficit is 37.3 mwc (for p_{min} = 20 mwc).

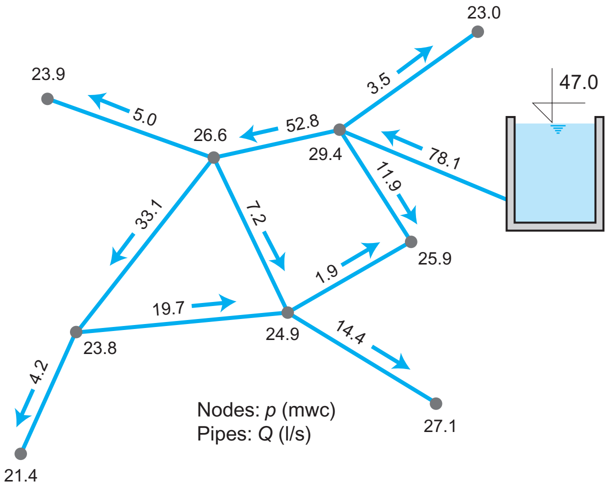


Figure A1.12. Pipe flows and nodal pressures – Problem A1.4.2.

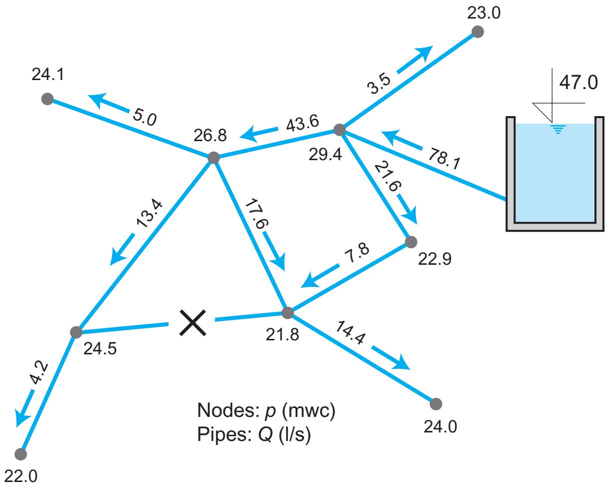


Figure A1.13. Pipe flows and nodal pressures – Problem A1.4.3a.

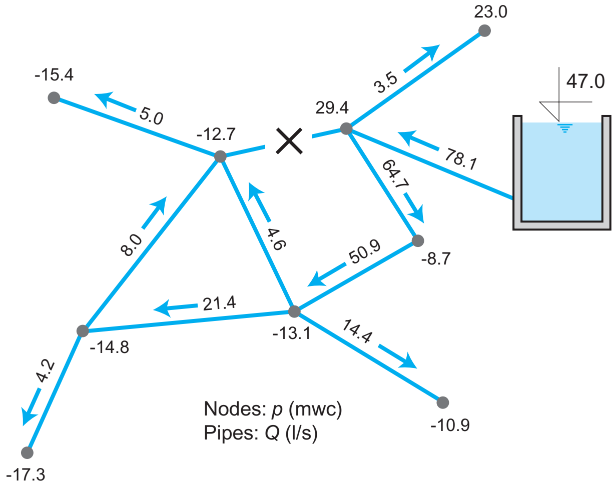


Figure A1.14. Pipe flows and nodal pressures – Problem A1.4.3b.

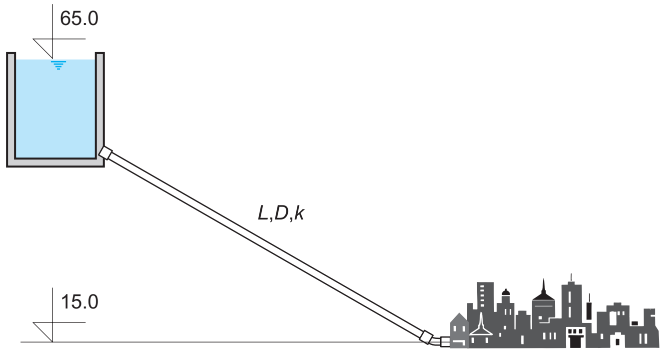


Figure A1.15. Distribution shceme – Problem A1.5.1.

A1.5 HYDRAULICS OF STORAGE AND PUMPS

PROBLEM A1.5.1

For the gravity system shown in Figure A1.15, find the maximum capacity of the transport pipe, when L = 3000 m, D = 800 mm and k = 0.5 mm, which can be delivered with a pressure of 35 mwc at the entrance of the city. Assume the water temperature to be 10°C.

Answer:
Q_{max} = 3782 m^3/h

PROBLEM A1.5.2

For the same system as in Problem A1.5.1, a pumping station is built next to the reservoir, as shown in Figure A1.16. The pump characteristics valid during the operation of all pumps is shown in Figure A1.17.

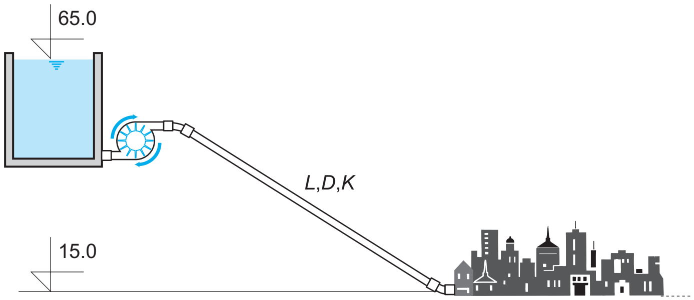


Figure A1.16. Distribution shceme – Problem A1.5.2.

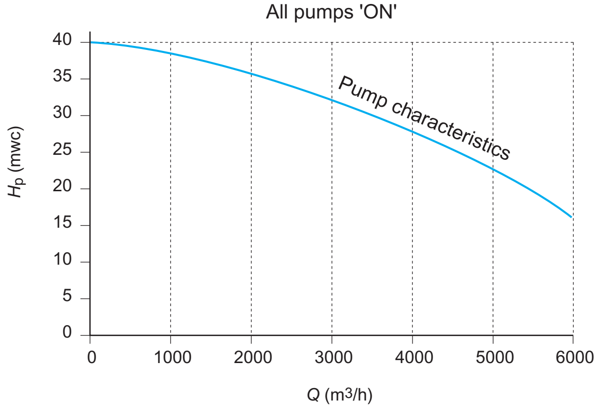


Figure A1.17. Pumping station Q/H curve – Problem A1.5.2.

Determine:

a the maximum flow of the transport system that can be delivered to the city with the same pressure as in Problem A1.5.1,

b the pressure at the entrance of the city if the pumping station delivers the same flow as in Problem A1.5.1.

Answers:

a From the graph in Figure A1.18, the pump delivers a maximum capacity of ± 5630 m^3/h. The pumping head of ± 18 mwc is used in this case to cover the friction loss increase.

b From the graph in Figure A1.19, the pump delivers a head of ± 29 mwc. As the entire friction loss is covered by gravity, the pumping head will be utilised to deliver the pressure at the entrance of the city. Thus, $p_{entr} = 35+29 = 64$ mwc.

The hydraulic grade lines for both modes of operation are shown in Figure A1.20.

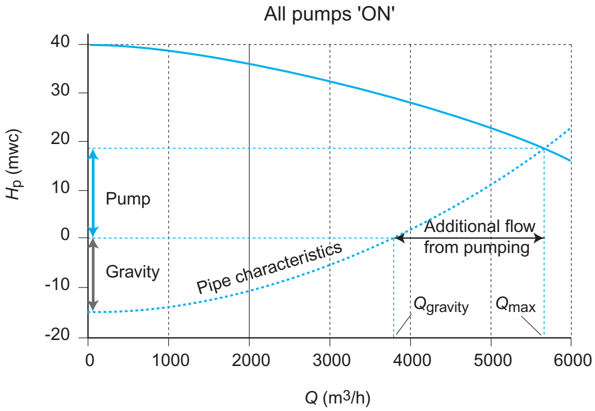


Figure A1.18. Pumping station operation – Problem A1.5.2a.

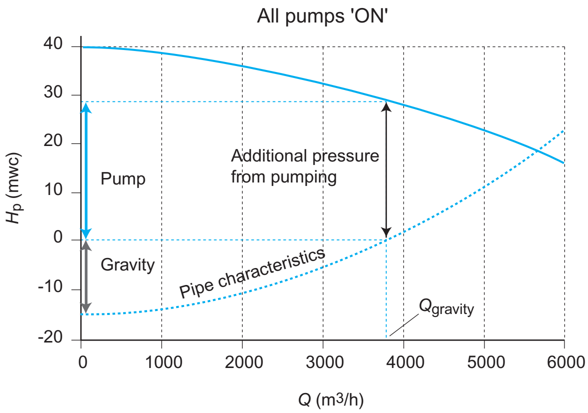


Figure A1.19. Pumping station operation – Problem A1.5.2b.

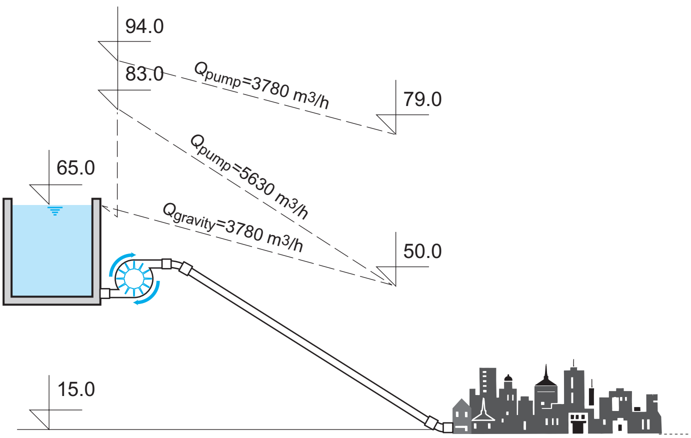


Figure A1.20. Hydraulic grade lines – Problem A1.5.2.

PROBLEM A1.5.3

For the combined system shown in Figure A1.21, find the maximum capacity and corresponding pressure at the entrance of the city. Avoid negative pressures along the route. The pipes are:

A–B: $L = 2000$ m, $D = 600$ mm, $k = 1.0$ mm,

B–C: $L = 1200$ m, $D = 700$ mm, $k = 0.1$ mm.

The pumping station operates according to the curve in Figure A1.17. The water temperature may be assumed to be10°C.

Note:

The theoretical maximum flow, without negative pressures, is reached for $\Delta H_{A-B} = 50 - 25 = 25$ mwc.

Answer:

$Q_{max} = 2596$ m^3/h. The pumping head for this flow is ± 34 mwc. Consequently, the calculated $\Delta H_{B-C} = 4.3$ mwc leads to a p_C of 19.7 mwc.

PROBLEM A1.5.4

For the system shown in Figure A1.22, determine the pressure at the entrance of the city for a flow of 800 m^3/h. The pipes are as follows:

A–B: $L = 1500$ m, $D = 500$ mm, $k = 0.5$ mm,

B–C: $L = 1200$ m, $D = 400$ mm, $k = 0.1$ mm.

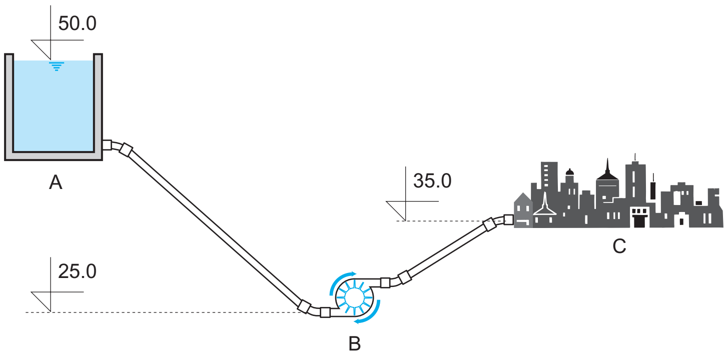


Figure A1.21. Distribution scheme – Problem A1.5.3.

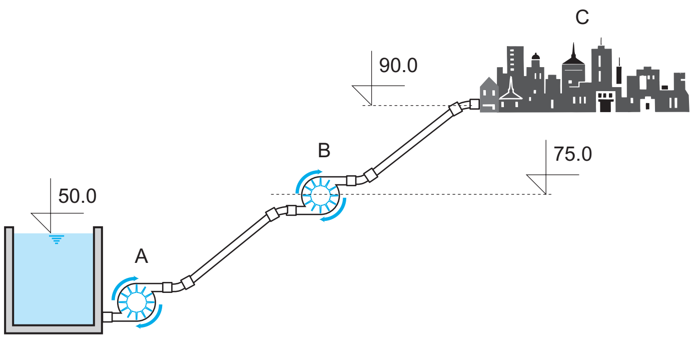


Figure A1.22. Distribution scheme – Problem A1.5.4.

Both pumping stations in A and B operate according to the curve shown in Figure A1.23. The water temperature may be assumed to be 10° C.

Answer:
For Q = 800 m^3/h, the total pumping head H_p = 70.4 mwc. $\Delta H_{A\text{–}B}$ + $\Delta H_{B\text{–}C}$ = 4.0 + 7.6 = 11.6 mwc. Thus, p_C = 18.8 mwc.

PROBLEM A1.5.5
For the system shown in Figure A1.24, determine the maximum flow that can be pumped from reservoir A to reservoir B. If the same capacity has to be transported by gravity, find the pressure at the entrance of the city. The pipes are as follows:

A–B: L = 1350 m, D = 450 mm, k = 0.1 mm,
B–C: L = 1800 m, D = 500 mm, k = 0.1 mm.

The pumping station operates according to the curve shown in Figure A1.23. The water temperature may be assumed to be 10° C.

Answer:
The maximum pumping capacity Q = 805 m^3/h (see Figure A1.25). This flow is delivered by gravity when pressure p_C = 21.2 mwc.

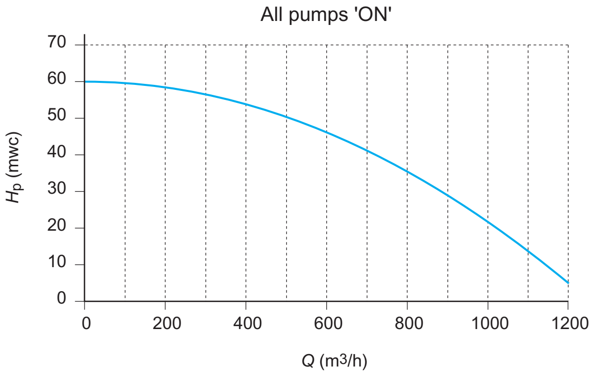


Figure A1.23. Pumping station Q/H curve – Problem A1.5.2.

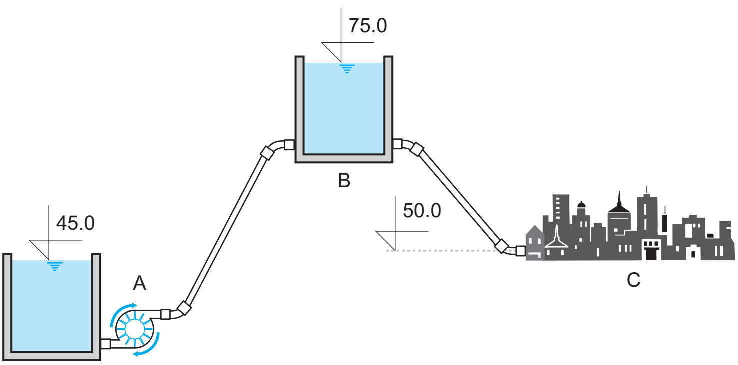


Figure A1.24. Distribution scheme – Problem A1.5.5.

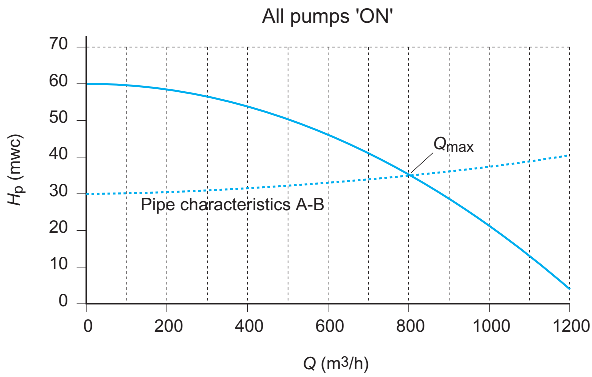


Figure A1.25. Pump operation at section A–B Problem A1.5.5.

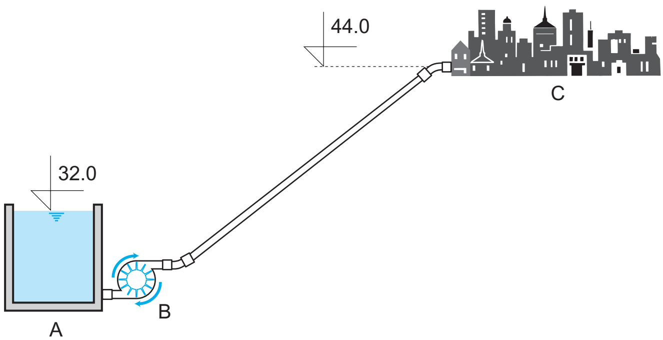


Figure A1.26. Distribution scheme – Problem A1.5.6.

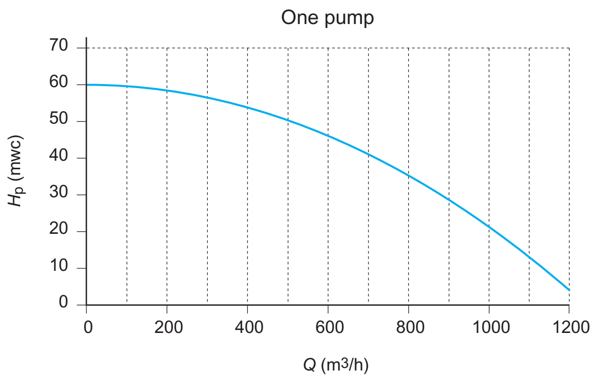


Figure A1.27. Pump characteristics – Problem A1.5.6.

PROBLEM A1.5.6

Pumping station B in Figure A1.26 supplies distribution area C from reservoir A through a pipe, where L = 1000 m, D = 600 mm, k = 1 mm. The pump characteristics of one pump unit are shown in Figure A1.27. The water temperature may be assumed to be 10°C.

The demand of the distribution area registered on the maximum consumption day was 28,008 m^3. The demand variation pattern during 24 hours is given in the following table:

Hour	1	2	3	4	5	6	7	8	9	10	11	12
pf_h	0.28	0.30	0.33	0.51	1.07	1.32	1.31	1.38	1.40	1.39	1.36	1.22
Hour	13	14	15	16	17	18	19	20	21	22	23	24
pf_h	1.07	1.04	1.12	1.18	1.29	1.37	1.30	1.33	1.06	0.60	0.44	0.33

Determine:

a the balancing volume of the reservoir assuming a constant (average) inflow over 24 hours,

b the required number of pumps arranged in parallel, which can provide the minimum-required pressure at the entrance of the city, i.e. p_{min} = 30 mwc, during the maximum consumption hour,

c the same as in ‘b’ but for the minimum consumption hour instead,

d the excessive pumping energy during the maximum and minimum consumption hours; the overall efficiency of the pumping station $\eta_{pst} = 0.65$.

Answers:

a $Q_{avg} = 1167$ m^3/h, $V_{bal} = 4.21Q_{avg} = 4913$ m^3.

b The maximum demand occurs at 9 a.m., when $Q_9 = 1.40 \times 1167 = 1634$ m^3/h. For this flow, $\Delta H_9 = 4.97 \approx 5$ mwc. The head required by one pump, $H_{p,9} = 12+5+30 = 47$ mwc, is reached for a flow of $\pm$ 580 m^3/h. Thus, three pumps are necessary.

c The minimum consumption occurs at 1 a.m., when $Q_1 = 0.28 \times 1167 = 327$ m^3/h. $\Delta H_1 = 0.2$ mwc, $H_{p,1} = 12+0.2+30 = 42.2$ mwc, for $Q_{p,1} = \pm 680$ m^3/h. Hence, one pump is sufficient.

d The actual pumping head during the maximum consumption hour is $\pm$48.5 mwc (for the flow 1634/3 = 544.7 m^3/h). The excessive head is 1.5 mwc and the wasted energy $E_w = 3.43$ kWh per single unit. Hence, for three units $E_w = 10.28$ kWh. During the minimum supply conditions, a flow of 327 m^3/h will be pumped against a head of $\pm$56 mwc. Thus, the excessive head is 13.8 mwc and the wasted energy $E_w = 18.92$ kWh when there is one unit in operation.

PROBLEM A1.5.7

Pumping station B in Figure A1.28 supplies the distribution area C from reservoir A through a pipe $L = 1100$ m, $D = 250$ mm, $k = 0.05$ mm. The pump characteristics of one pump unit is shown in Figure A1.29. The minimum-required pressure at the entrance of the city is 25 mwc. The water temperature may be assumed to be 10° C.

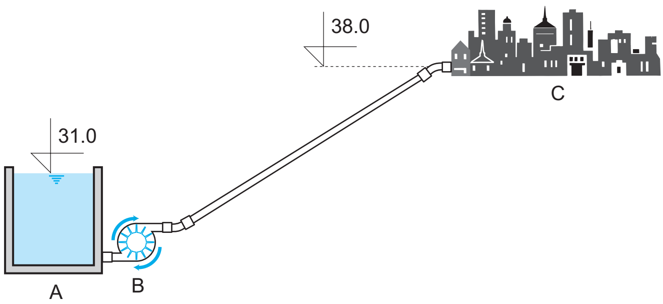


Figure A1.28. Distribution scheme – Problem A1.5.7.

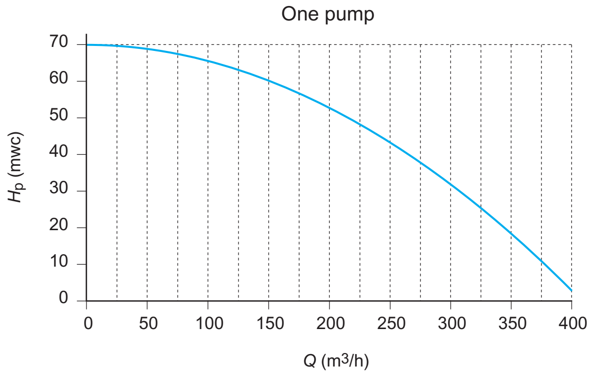


Figure A1.29. Pump characteristics – Problem A1.5.7.

Determine:

a the maximum flow that can be supplied when one pump is in operation,

b the maximum flow that can be supplied if two parallel pumps are in operation,

c the maximum flow that can be supplied in cases 'a and b', if another pipe with D = 250 mm is laid in parallel.

Answers:

a Q_{max} = 260 m^3/h (see Figure A1.30).

b Q_{max} = 410 m^3/h (see Figure A1.31).

c By laying the second pipe where D = 250 mm, each pipe will transport half of the initial flow, which reduces the friction losses. The composite system characteristics is shown in Figure A1.32. From the graph: $Q_{max,1}$ = 280 m^3/h for one pump in operation, and $Q_{max,2}$ = 520 m^3/h, for two pumps.

PROBLEM A1.5.8

Distribution area C in Figure A1.33 is supplied by gravity through a pipe where L = 750 m, D = 500 mm, k = 0.5 mm. The volume of reservoir

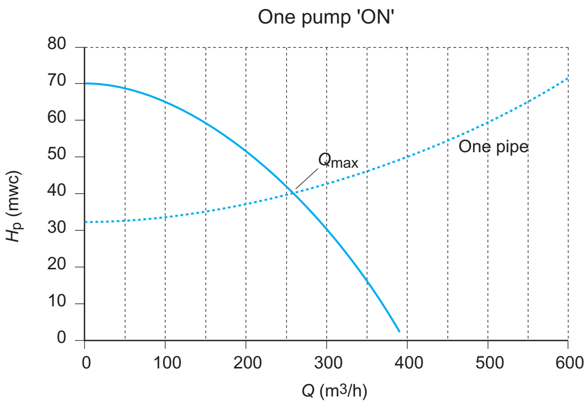


Figure A1.30. Pump operation, one pump – Problem A1.5.7a.

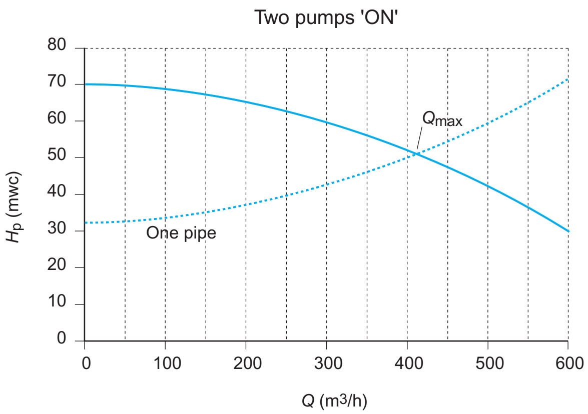


Figure A1.31. Pump operation, two pumps and one pipe – Problem A1.5.7b.

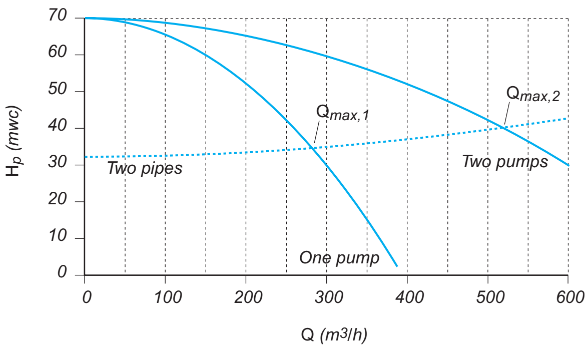


Figure A1.32. Pump operation, two pumps and parallel pipes– Problem A1.5.7c.

B is recovered by pumping A from a well field. The water temperature $T = 10°$ C.

The demand of the distribution area that was registered on the maximum consumption day was 30,480 m^3. The demand variation

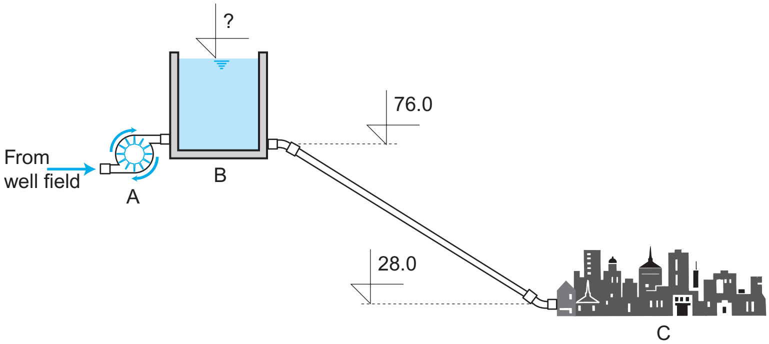


Figure A1.33. Distribution scheme – Problem A1.5.8.

pattern during 24 hours is given in the following table:

Hour	1	2	3	4	5	6	7	8	9	10	11	12
pf_h	0.71	0.75	0.77	0.79	0.96	1.14	1.15	1.18	1.20	1.19	1.17	1.07
Hour	13	14	15	16	17	18	19	20	21	22	23	24
pf_h	0.96	0.94	1.00	1.04	1.12	1.17	1.16	1.14	0.96	0.87	0.79	0.76

Determine:

a the balancing volume of the reservoir, assuming constant (average) pumping over 24 hours,

b the 24-hour water level variation in the tank, assuming the tank has a cross-section area of 1000 m^2 and provision for all other purposes of 60% of the total volume,

c the range of pressures that appear over 24 hours at the entrance of the town.

Answers:

a $Q_{avg} = 1270$ m^3/h, $V_{bal} = 1.64Q_{avg} = 2085$ m^3

b The total volume $V_{tot} = 5215$ m^3. The available tank depth is 5.22 m.

Hour	pf_{out}	pf_{in}	$pf_{in} - pf_{out}$	Σ	Depth (m)
1	0.71	1	0.29	0.29	4.29
2	0.75	1	0.25	0.54	4.61
3	0.77	1	0.23	0.77	4.90
4	0.79	1	0.21	0.99	5.17
5	0.96	1	0.04	**1.02**	**5.22**
6	1.14	1	−0.14	0.88	5.04
7	1.15	1	−0.15	0.73	4.85
8	1.18	1	−0.18	0.55	4.62
9	1.20	1	−0.20	0.36	4.37
10	1.19	1	−0.19	0.17	4.13
11	1.17	1	−0.17	0.00	3.92

12	1.07	1	−0.07	−0.07	3.83
13	0.96	1	0.04	−0.04	3.87
14	0.94	1	0.06	0.02	3.94
15	1.00	1	0.00	0.02	3.94
16	1.04	1	−0.04	−0.02	3.89
17	1.12	1	−0.12	−0.14	3.74
18	1.17	1	−0.17	−0.32	3.52
19	1.16	1	−0.16	−0.48	3.31
20	1.14	1	−0.14	**−0.62**	**3.13**
21	0.9	1	0.04	−0.58	3.18
22	0.87	1	0.13	−0.46	3.34
23	0.79	1	0.21	−0.24	3.61
24	0.76	1	0.24	0.00	3.92

At midnight, $V_0 = 0.6V_{tot} + 0.62Q_{avg} = 3917$ m^3 that corresponds to the depth of 3.92 m (see Figure A1.34).

c The maximum demand occurs at 9 a.m., where $Q_9 = 1524$ m^3/h. For this flow, $\Delta H_9 = 7.14$ mwc and the remaining pressure in C, $p_{C,9} = 76 + 4.37 - 7.14 - 28 \approx 45$ mwc. For the minimum at 1 a.m., $Q_1 = 902$ m^3/h. $\Delta H_1 = 2.54$ mwc and $p_{C,1} \approx 50$ mwc.

PROBLEM A1.5.9

For the same problem as in A1.5.8 determine the balancing volume of the reservoir assuming constant pumping of twice the average flow during 12 hours:

a between 8 p.m. and 8 a.m.,

b from 9 a.m. to 3 p.m. and from 11 p.m. until 5 a.m.

Answers:

a $V_{bal} = 13.21Q_{avg} = 16{,}780$ m^3

b $V_{bal} = 8.47Q_{avg} = 10{,}760$ m^3.

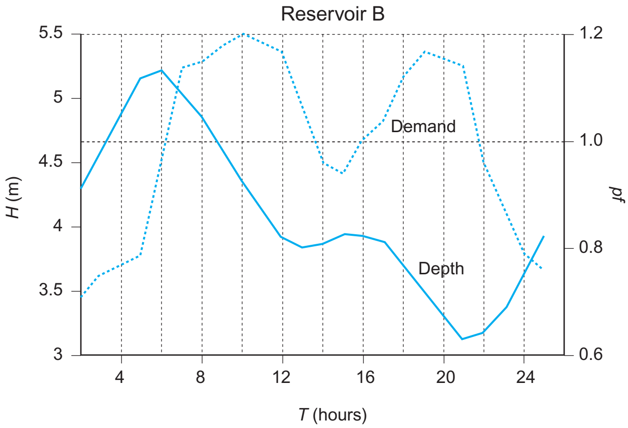


Figure A1.34. Water variation vs. demand pattern – Problem A1.5.8.

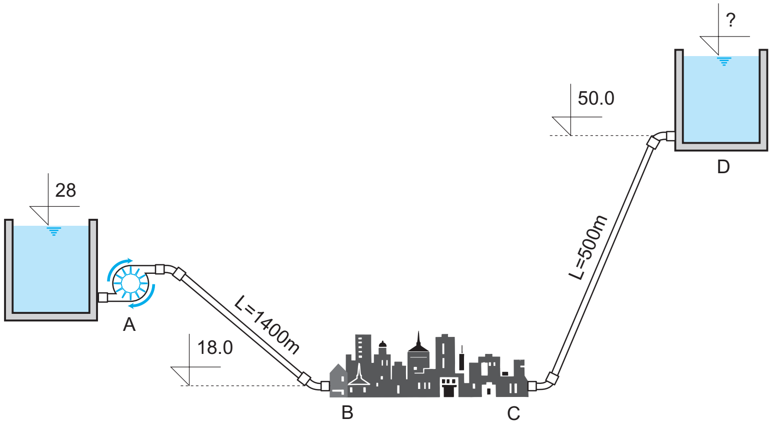


Figure A1.35. Distribution scheme – Problem A1.5.10.

PROBLEM A1.5.10

A distribution area is supplied, as shown in Figure A1.35. The following is a typical demand variation:

Hour	1	2	3	4	5	6	7	8	9	10	11	12
pf_h	0.79	0.82	0.83	0.83	0.94	1.06	1.09	1.08	1.11	1.12	1.11	1.05
Hour	13	14	15	16	17	18	19	20	21	22	23	24
pf_h	1.02	1.00	1.04	1.07	1.08	1.08	1.09	1.09	1.01	0.96	0.89	0.83

On the maximum consumption day, the pumping station was working at constant (average) capacity of 1800 m^3/h against a head of 34 mwc. Determine:

a the balancing volume of the tank located in point D,

b the 24-hour water level variation in the tank, assuming the tank has a cross-section area of 1650 m^2 and there is provision for all other purposes of 70% of the total volume,

c pipe diameters A–B and C–D, providing the minimum pressures at B and C of 35 mwc, during the maximum consumption hour,

d pressures during the minimum consumption hour, required to refill the tank.

For both pipes $k = 0.5$ mm. Assume the water temperature to be 10°C.

Answers:

a $V_{bal} = 1.1Q_{avg} = 1980\ m^3$

b The total volume $V_{tot} = 6600\ m^3$. The available tank depth is 4.0 m. At midnight, $V_0 = 0.7V_{tot} + 0.32Q_{avg} = 5196\ m^3$ when the depth is 3.15 m (see Figure A1.36).

c The maximum demand occurs at 10 a.m., $Q_{10} = 2016\ m^3/h$, while the minimum appears at 1 a.m., $Q_1 = 1422\ m^3/h$. In both cases the

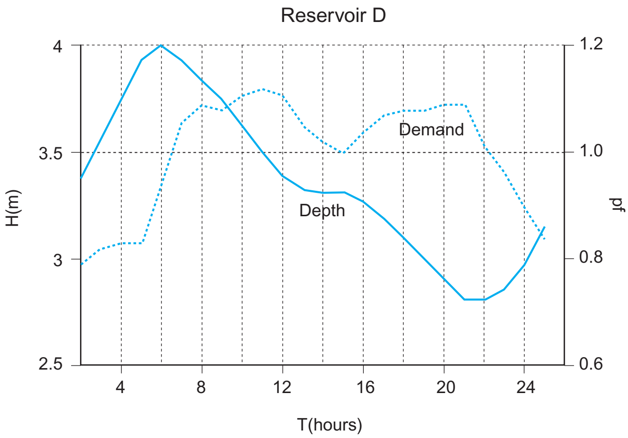


Figure A1.36. Water depth variation in tank vs. demand pattern – Problem A1.5.10.

pumping station supplies 1800 m^3/h and the difference comes from, or goes into, the tank. Hence, Q_{A-B} = 1800 m^3/h, H_{A-B} = (28 + 34) – (18 + 35) = 9 mwc. The optimum diameter for these conditions is D_{A-B} = 574 mm (manufactured diameter = 600 mm). At 10 a.m., the tank supplies the system with 216 m^3/h from the elevation 50 + 3.50 = 53.50 msl. ΔH_{D-C} = 53.5 – (18 + 35) = 0.5 mwc. The optimum diameter for such conditions is D_{D-C} = 369 mm (manufactured at 400 mm).

d During the minimum demand hour, the tank receives 378 m^3/h to the elevation of 50 + 3.38 = 53.38 msl. $\Delta H_{C-D} \approx$ 1.0 mwc for D = 400 mm. Thus, the required pressure in C, p_C = 53.4 + 1 − 18 = 36.4 mwc. For D_{A-B} = 600 mm, ΔH_{A-B} = 7.2 mwc and p_B = 36.8 mwc $> p_C$. Hence, neglecting the resistance in the distribution area itself would allow water to reach the tank. However, as this is not reasonable to assume, either a higher pumping head in A or an additional booster station is needed.

APPENDIX 2

Design Exercise

The main objective of this exercise is to analyse the hydraulic performance of looped water distribution networks with the help of a computer-modelling tool; EPANET software developed by the US Environmental Protection Agency is used throughout the tutorial. For this purpose, a model of the case network has been prepared, which consists of a group of EPANET input files describing the various steps of the analysis. All file names follow uniform coding: 'Sxyz.NET', where:

- S stands for Safi (the name of the hypothetical town used in this case),
- $x = 6$ or 30, which are respectively the years of the beginning and the end of the design period,
- $y =$ A or B, which are the two alternatives requested by the assignment,
- $z =$ the serial number of the particular network layout explored within the alternative.

The results of calculations are presented in the tables and figures. These may be accompanied by:

- *Clarification* that explain some (calculation) details from the table/figure,
- *Conclusions* that suggest further steps based on the calculation results,
- *Comments* that elaborate points in a wider context of the problem,
- *Reading* that refers to the chapters of the main text that are related to the problem.

Although the explanations in the tutorial should be understood without necessarily using the computer programme, more will be gained if the case network is studied in parallel on a PC. For this purpose, the full version of the EPANET 2 software and the case network files mentioned in the text are enclosed on the attached CD. General information relevant for this exercise, which includes the programme installation and digested instructions for use, is given in Appendix 6.

A2.1 CASE INTRODUCTION – THE TOWN OF SAFI

Safi is a town of approximately *160,000 inhabitants*. Over the coming years, a rapid expansion of the area around the town is expected because of good regional development activities. A factory for production of agricultural machinery is going to be built near the town itself.

The existing water distribution network, which presently covers just a half of the area of the town, is in a very poor condition and the construction of a completely new system has been proposed. The pumping station at the source is also rather old and of insufficient capacity. Its general overhaul has also been planned for during the project implementation. An international loan for both purposes was requested and has been approved.

A looped-type network is planned for the construction. The planned design period of the new distribution network is *25 years*. Within that time, the system should provide sufficient amounts of water and pressure variations of between *20* and *60 mwc* during regular operation. Based on the evaluation of the present situation and the estimates for future growth, a technically and economically acceptable solution for construction of the network has to be proposed.

A2.1.1 *Topography*

Safi is a secondary town located in a hilly area with ground elevations ranging between *8 and 28 msl*. Figure A2.2 shows the configuration of the terrain together with the layout of the main streets in the urban area (dotted lines).

A2.1.2 *Population distribution and future growth*

The town is divided into six districts (A–F, in Figure A2.1), each of a homogenous population density. The total surface areas of the districts and sub-districts are given in Table A2.1.

The present population density for each area, expressed as the number of inhabitants per hectare, is shown in Table A2.2.

In the absence of more reliable data, the indicated population growth is assumed to be constant throughout the entire design period.

A2.1.3 *Supply source*

The source of supply located northwest from the town (*Source 1* shown in Figure A2.2) is groundwater stored after simple treatment in a ground reservoir from where it is pumped to the system.

The present pumping station at the source is old and a new one is going to be built, together with a clear water reservoir at the suction side.

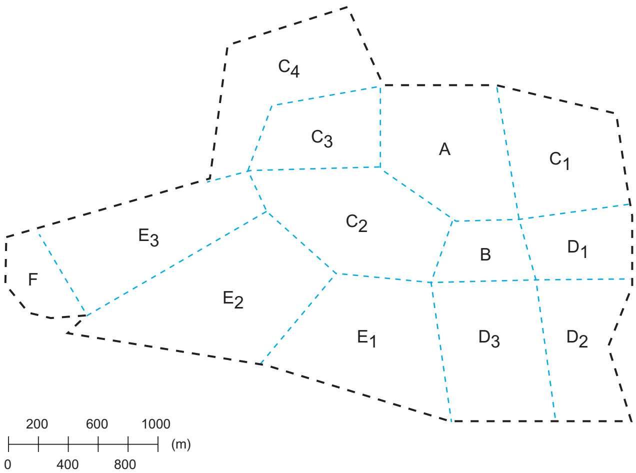


Figure A2.1. Areas of different population density.

Table A2.1. The town of Safi: surface area of the districts and sub-districts.

District	Area (ha)	District	Area (ha)	District	Area (ha)
A	83.2	C_3	50.0	D_3	78.0
B	30.8	C_4	84.4	E_1	97.2
C_1	73.6	D_1	37.2	E_2	117.2
C_2	80.8	D_2	55.6	E_3	83.2
				F	29.6

Table A2.2. The town of Safi: population density and growth.

District	Present (inhab./ha)	Annual growth(%)
A	225	1.0
B	248	1.2
$C_{1,2,3,4}$	186	2.0
$D_{1,2,3,4}$	202	2.7
$E_{1,2,3}$	144	2.9
F	115	3.3

Centrifugal, fixed speed pumps are going to be installed in a parallel arrangement. The number and size of the pumps should be sufficient to provide smooth operation of the system, which includes a number of anticipated irregular supply scenarios.

The maximum production capacity of the present source is *2500 m³/h*. The additional ground water source located south (*Source 2*) can be developed in the future, if the indicated capacity is exceeded.

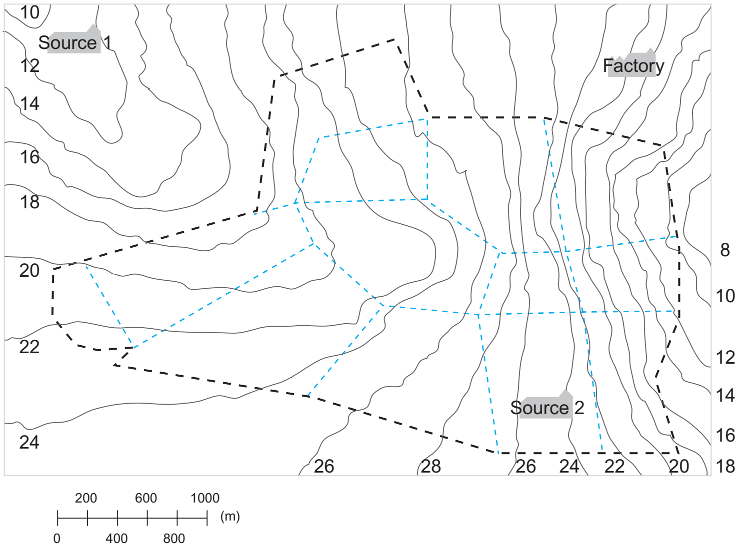


Figure A2.2. Topography and location of supply sources.

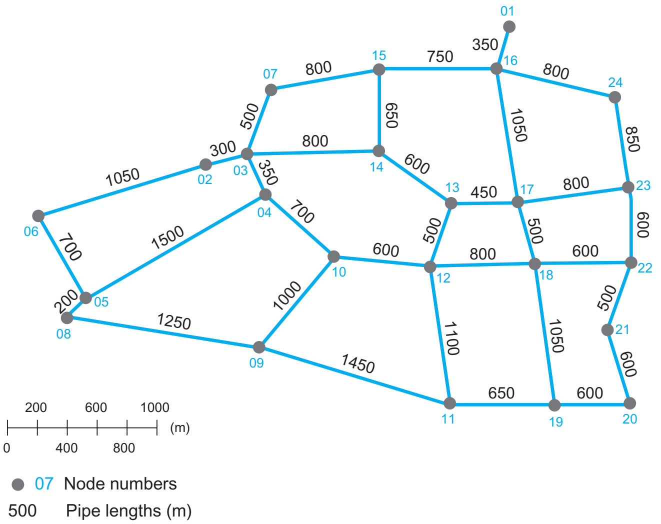


Figure A2.3. Node numbers and pipe routes.

A2.1.4 *Distribution system*

The distribution pipes will be laid alongside the main streets of Safi. The network layout is shown in Figure A2.3, with the lengths of the pipe routes. The intersections (nodes) are numbered. Connection of the factory (Node 01) to the system is planned to be in Node 16.

Regarding the local situation (soil conditions, local manufacturing), PVC has been chosen as a pipe material for the network. Because of this choice, it is assumed that the roughness of the pipe will remain low throughout the design period. The accepted *k*-value of *0.5 mm* includes the impact of local losses in the network.

A2.1.5 *Water demand and leakage*

The water consumption in the area is mainly domestic, except for the new factory that will also be supplied from the distribution system. In the new system, the domestic consumption per capita is planned to be *150 l/d*. In addition, the projected water demand of the new factory is *1300* m^3/d. The unaccounted-for water (UFW) is not expected to exceed *10%* of the water production after the commissioning of the system. This is assumed to increase by a further *10%* by the end of the design period, i.e. would grow to *20%* of the water production.

The typical diurnal domestic consumption pattern is shown in Figure A2.4, which does not include UFW. Variation of the consumption during the week is within the range of *0.95* and *1.10* of the average daily figure. In addition, the monthly (seasonal) variations are in the annual range of between *90%* and *115%* of the average consumption. The factory will be working at a constant (average) capacity for *12 hours a day*, between *7 a.m.* and *7 p.m.* (two shifts). Neither the seasonal variations nor the UFW percentage applies in this case.

A2.1.6 *Financial elements*

An international loan has been requested and approved with a payback period of *30 years* and an interest rate of *8%*. The loan is intended only for construction of the distribution part of the network, which does not include the tertiary network or service connections.

The monthly instalments are to be paid entirely through the sale of water. It is therefore agreed that the re-payments will start only after the system has been commissioned, i.e. five years from now.

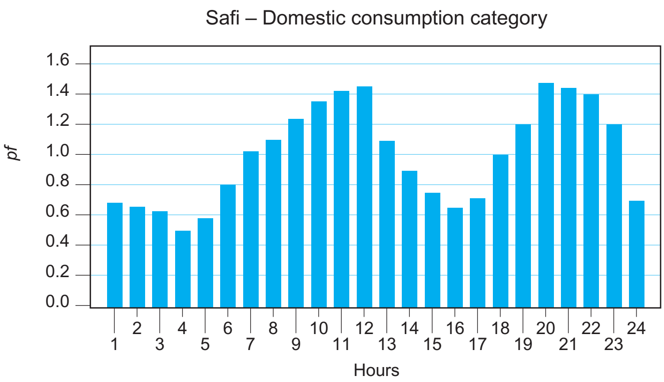


Figure A2.4. Consumption pattern.

Table A2.3. Investment costs of the equipment.

No.	Component	US$	per
1.1	Laying pipe D = 80 mm	60.–	m
1.2	Laying pipe D = 100 mm	70.–	m
1.3	Laying pipe D = 150 mm	90.–	m
1.4	Laying pipe D = 200 mm	130.–	m
1.5	Laying pipe D = 300 mm	180.–	m
1.6	Laying pipe D = 400 mm	260.–	m
1.7	Laying pipe D = 500 mm	310.–	m
1.8	Laying pipe D = 600 mm	360.–	m
2	Pumping station	$0.5 \times 10^4 \times Q^{0.8}$	$Q = Q_{max}$ m³/h
3	Reservoir	$35 \times 10^4 + 150 \times V$	$V = V_{tot}$ m³
4	Support structure for H m-elevated tank	$3 \times H \times V$	$V = V_{tot}$ m³

Table A2.4. Annual operation and maintenance costs.

No.	Component	% of investment
1	Distribution pipes	0.5
2	Pumping station	2.0
3	Storage	0.8

Procurement of the equipment will be done according to the prices listed in Table A2.3. If considering the possibility of laying of pipes in parallel, the price indicated under 1.1–1.8 should be doubled.

Operation and maintenance costs are determined as a percentage of the investment costs (Table A2.4):

As all cost calculations are to be made in US$, the effect of local inflation can be ignored.

A2.2 QUESTIONS

A2.2.1 *Hydraulic design*

Preliminary concept

1.1 Calculate the demand increase throughout the design period.

Two possible alternatives for the network design have to be analysed:

A Supply by direct pumping (source pumping stations)

B Supply by pumping and by gravity (balancing tank or water tower)

For both alternatives:

1.2 Regarding the supply points:

- determine if and when the second source will need to be put into operation and calculate its maximum required capacity by the end of the design period,

- decide in which nodes the sources need to be connected to the network, and find out the lengths of the connecting pipes.

1.3 Develop a preliminary supply strategy: suggest possible phases in the development of the system, function of the pumping stations, reservoirs, etc.

Nodal consumptions

1.4 Calculate the average consumption for nodes 01–24
- at the beginning of the design period,
- at the end of the design period.

Network layout

1.5 Size the pipe diameters in the system at the beginning and the end of the design period.

Pumping stations

1.6 For the source pumping stations, determine:
- the required duty head and duty flow,
- the provisional number and arrangement of the pumping units, both at the beginning and the end of the design period.

1.7 Determine the location, duty head and flow of the booster station(s), if needed in the system.

Storage

1.8 Determine the volume of the clear water reservoir at the suction side of the source pumping stations, at the beginning and the end of the design period (assume ground level tanks with a bypass for maintenance purposes).

1.9 For Alternative B only: determine the provisional location, volume and dimensions of the balancing storage in the network at both the beginning and the end of the design period.

Summary

1.10 Draw conclusions based on the hydraulic performance of the system throughout the entire design period. Explain the phased development of the system, if applied. Suggest a preference for the A- or B-layout.

A2.2.2 *System operation*

Upgrade the computer model of the selected layouts A and B at the end of the design period. Model the pumping station operation by assuming a parallel arrangement of the pump units.

For both alternatives, run the simulation of the maximum consumption day and consider the following steps:

Regular operation

2.1 Propose a plan of operation for both pumping stations during regular supply conditions. Compare the manual pump operation with an automatic operation based on the pressure in selected critical node (or tank) in the network.

Factory supply under irregular conditions

2.2 A reliable supply and fire protection have to be provided for the factory. Analyse the preliminary layout under the maximum consumption hour conditions, if:
- a single pipe, either 15–16, 16–24 or 16–17, bursts,
- a requirement for fire fighting of *180* m^3/h at a pressure of *30 mwc* is needed in Node 01.

If necessary, propose an operation that can provide a supply of the required quantities and pressures.

Network reliability assessment

2.3 Assess the reliability of the distribution network. Determine the consequences of single pipe failure events if a supply of minimum *75%* of the maximum consumption hour demand has to be maintained in the system.

Choice of final layouts

Summarise all the findings from the exercise and adjust the network layouts for alternatives A and B, where required. For these final layouts

2.4 Show the network layout, number and size of the pump units and distribution of the storage volume at the beginning of the design period.
2.5 For the manual mode of operation on the maximum consumption day, show the range of pressures in the system. For Alternative B only, show the volume variation in the balancing tank.
2.6 Describe the steps for the reconstruction/extension of the system and show its operation at the end of the design period (as required by Question 2.5).

Cost comparisons

2.7 Calculate the investment costs of the network.
2.8 Calculate the operation and maintenance (O and M) costs of the network.
2.9 Determine the average increase in cost per cubic meter of water, due to the loan repayment and O and M of the system.

If a phased development of the system is planned, explain its impact on the above costs.

Summary
2.10 Summarise all the conclusions and select the final design.

A2.3 HYDRAULIC DESIGN

A2.3.1 *Preliminary concept*

By combining the information from Tables A2.1 and A2.2, the population and domestic consumption figures are shown in Table A2.5. A spreadsheet calculation is conducted for:

- the present situation (Year 1),
- commissioning of the system after five years (Year 6),
- the end of the 25-year design period (Year 30).

Clarification:

- An exponential growth model is assumed in the demand calculations.

Conclusions:

- The population i.e. the demand will nearly double in the next 30 years.

Table A2.5. Population and domestic consumption growth.

District	Area (ha)	Density (inh./ha)	Growth (%)	Population (inhabitants) Present	At year 6	At year 30
A	83.2	225	1.0	18,720	19,872	25,232
B	30.8	248	1.2	7638	8205	10,925
C_1	73.6	186	2.0	13,690	15,417	24,797
C_2	80.8	186	2.0	15,029	16,925	27,223
C_3	50.0	186	2.0	9300	10,473	16,846
C_4	84.4	186	2.0	15,698	17,679	28,435
D_1	37.2	202	2.7	7514	8817	16,711
D_2	55.6	202	2.7	11,231	13,178	24,977
D_3	78.0	202	2.7	15,756	18,487	35,040
E_1	97.2	144	2.9	13,997	16,616	32,998
E_2	117.2	144	2.9	16,877	20,035	39,788
E_3	83.2	144	2.9	11,981	14,223	28,245
F	29.6	115	3.3	3404	4136	9016
Total				**160,835**	**184,062**	**320,232**
Specific consumption (l/c/d)			150			
Total average consumption						
m^3/d				24,125	27,609	48,035
m^3/h				1005.2	1150.4	2001.5
l/s				279.23	319.55	555.96

Comments:

- The exponential growth model simulates a faster growth of population than the linear growth model does. Local conditions determine which of the two models is more suitable.

Reading:

- Chapter 2, Paragraph 2.5: ‘Demand forecasting’.

The design alternatives to be compared are:
A Supply by direct pumping from the source(s)
B Supply combined by pumping and by gravity

The principal difference between these two is:
A The entire demand in the area is supplied from the source pumps at all times. Moreover, under extreme conditions the sources must be capable of satisfying the demand during the maximum consumption hour of the maximum consumption day.
B In peak periods of the day, part of the demand can be supplied from the balancing tank in the system. The source will normally supply the average flow on the maximum consumption day, provided the position and volume of the balancing tank are properly determined.

Consequently, the first source will reach its maximum capacity of 2500 m^3/h sooner in the case of Alternative A than in Alternative B. Table A2.6 indicates when this will happen.

Clarification:

- The demand calculation in the table is based on Equation 2.1 in Chapter 2.
- The growth scenario of the UFW percentage is hypothetical.
- The maximum and minimum consumption day demands are determined based on the seasonal factors $1.10 \times 1.15 = 1.265$ and $0.95 \times 0.90 = 0.855$, respectively.
- The factory is treated as a major user. Unlike for the domestic demand, its demand (1300 m^3/d, constant throughout the design period) is included without taking into account any influences of the seasonal variations and UFW.
- The maximum/minimum consumption hour demands are calculated from the peak factors displayed in Figure A2.4. The factory demand is added based on a 12-hour operation (1300/12 = 108.3 m^3/h) if the factory was in operation during the maximum/minimum consumption hour.

Conclusions:

- From the domestic diurnal diagram, the maximum hourly peak factor is at 20:00 hours (1.47). Nevertheless, the maximum consumption

Table A2.6. Demand growth.

					Seasonal variations		Peak hour at:	
					Max	Min	12	4
	Factory:	1300	m^3/d	Factors:	1.265	0.855	1.45	0.5
	Average demands				Peak demands			
	Population (inh)	Consumption (m^3/d)	UFW (%)	Demand (m^3/d)	$Q_{max,day}$ (m^3/d)	$Q_{min,day}$ (m^3/d)	$Q_{max,hour}$ (m^3/h)	$Q_{min,hour}$ (m^3/h)
Present	160,835	24,125	0	24,125	30,518	20,627	1843.8	429.7
Year 06	184,062	27,609	10	31,977	40,106	27,529	2452.9	546.4
Year 07	188,274	28,241	10	32,679	40,994	28,129	**2506.5**	558.9
Year 08	192,590	28,889	10	33,398	41,904	28,744	2561.5	571.8
Year 09	197,013	29,552	10	34,136	42,837	29,374	2617.9	584.9
Year 10	201,545	30,232	10	34,891	43,792	30,020	2675.6	598.3
Year 11	206,190	30,929	11	36,051	45,260	31,012	2764.3	619.0
Year 12	210,951	31,643	11	36,854	46,275	31,698	2825.6	633.3
Year 13	215,830	32,375	11	37,676	47,315	32,401	2888.4	647.9
Year 14	220,830	33,125	11	38,519	48,381	33,122	2952.8	663.0
Year 15	225,955	33,893	11	39,382	49,474	33,860	3018.9	678.3
Year 16	231,208	34,681	13	41,163	51,727	35,383	3155.0	710.1
Year 17	236,593	35,489	13	42,092	52,902	36,177	3225.9	726.6
Year 18	242,112	36,317	13	43,043	54,105	36,991	3298.7	743.6
Year 19	247,769	37,165	13	44,019	55,339	37,825	3373.2	760.9
Year 20	253,568	38,035	13	45,019	56,604	38,679	3449.6	778.7
Year 21	259,513	38,927	16	47,642	**59,922**	40,922	3650.1	825.5
Year 22	265,607	39,841	16	48,730	61,299	41,852	3733.3	844.8
Year 23	271,855	40,778	16	49,846	62,710	42,806	3818.5	864.7
Year 24	278,259	41,739	16	50,989	64,157	43,784	3905.9	885.1
Year 25	284,826	42,724	16	52,162	65,640	44,787	3995.6	906.0
Year 26	291,558	43,734	20	55,967	70,454	48,040	4286.4	973.8
Year 27	298,460	44,769	20	57,261	72,091	49,147	4385.3	996.8
Year 28	305,536	45,830	20	58,588	73,769	50,281	4486.7	1020.4
Year 29	312,792	46,919	20	59,949	75,490	51,444	4590.7	1044.7
Year 30	320,232	48,035	20	61,344	**77,255**	52,637	**4697.3**	1069.5

hour occurs at 12:00 hours due to the factory demand (combined with the peak factor of the domestic demand of 1.45). The minimum consumption hour is at 04:00 hours (peak factor = 0.50 and the factory is closed).

- By applying the direct pumping scheme (Alternative A), the maximum capacity of the first source is going to be reached in Year 7 i.e. already by the second year after being commissioned. The additional peak capacity needed from the second source is therefore: $4697.3 - 2506.5 \approx 2200$ m^3/h (see Table A2.6, column $Q_{max,hour}$).
- By applying the combined system (Alternative B), the capacity of the first source is going to be reached in Year 21. If this capacity is

supplied 24 hours a day (2500 m^3/h = 60,000 m^3/d), the additional capacity needed from the second source will be 77,255 − 59,922 ≈ 17,300 m^3/d (Table A2.6, column $Q_{max,day}$). The combined supply scheme assumes that the balancing tank will provide/accommodate the difference between any hourly flow and the average flow during 24 hours. This assumption is going to be taken as a starting point in this design alternative.

Comments:

- The calculation procedure applied in Table A2.6 is determined by the origin of the diurnal pattern.
- The crucial questions to be answered are:
  - Does the diurnal diagram represent one or more demand categories?
  - Does it include UFW (leakage) or not?
  - Which consumption day does it represent (average, maximum, minimum)?
- A distinction should be made between the UFW percentage and the leakage percentage (i.e. physical loss). Part of the UFW is water that is in the system but is not paid for, e.g. due to illegal connections or water meter under reading. Consequently, including the entire UFW level instead of the leakage level alone reduces the calculated pressures, which eventually works as a kind of safety factor for the design. Moreover, the UFW is often much easier to assess than its isolated components, as this figure is reflected in the loss of revenue.

Reading:

- Chapter 2, Paragraphs 2.3 and 2.4: ‘Water demand patterns’ and ‘Water Demand calculation’.

Scenarios

The following scenarios are going to be tested while developing the system:

Alternative A:

- Both sources 1 and 2 will be connected to the system at the beginning of the design period because the first source alone will become exhausted soon after the system is commissioned. By connecting the second source immediately, the whole construction can be carried out at once.
- The sources should have a similar capacity during regular operation of the system. They are located on the opposite sides of the system that allows for a network of similar pipe diameters, which is in principle a more convenient layout for operation (in irregular situations) and for future extensions.
- Source 1 will not exceed its maximum capacity during regular operation. Some 10–20% of the capacity will be kept as a stand-by for irregular supply (fire fighting, pipe bursts, etc.).

- In case of failure or planned maintenance in any of the two sources, the source remaining in operation should have sufficient capacity to supply the factory.

Alternative B:
- Source 1 has to supply the average flow on the maximum consumption day. The diurnal demand variation will therefore be satisfied from the balancing tank.
- The balancing tank should also have provision for irregular supply conditions.
- From the year in which it reaches its maximum capacity, the first source will continue to operate at this capacity until the end of the design period.
- The second source should become operational in Year 21, at the latest. If required, this source should also assist with supply during irregular situations.
- It is assumed that the volume of the balancing tank can only be recovered from the two sources (no other source in the vicinity of the tank is available).

In Alternative B, Source 1 is the major source of the system. The constant i.e. average delivery from this source is possible throughout the design period because the peak flows are to be supplied from the balancing tank. This also delays investment in Source 2. Once this source is put into operation, it will function as the second, or reserve, source.

In both alternatives:
- A looped structure for the secondary mains will be designed. Although more expensive than a branched one, it is a ‘must’ in any serious consideration of the system reliability (more reliability = more investment). Question 2.3 stipulates a requirement for high reliability in this assignment.
- Location of the source connections to the system will depend on the route for the secondary mains. Moreover
  - these connections should be laid towards the areas of higher demand,
  - the pipe route should be as short as possible,
  - the pipe should be easily accessible (passages underneath buildings should be avoided).

Reading:
- Chapter 3, Paragraph 3.7: ‘Hydraulics of storage and pumps’,
- Chapter 4, Sections 4.2.1: ‘Design criteria’ and 4.2.2: ‘Basic design principles’.

A2.3.2 *Nodal consumptions*

The average consumption in the nodes has been determined by calculating the specific consumption per metre of pipe in each loop, assuming an even dispersion of house connections throughout the system.

A spreadsheet calculation can be carried out based on the procedure described in Chapter 4, Paragraph 4.3: 'Computer models as design tools'. The nodal consumption is calculated for the following points of time: at Year 6 – the beginning of the design period, and at Year 30 – the end of the design period. The results are shown in the Table A2.7.

Clarification:

- The figures in the table represent averages, the so-called 'baseline' consumption. To determine actual demand at a certain moment, these figures have to be modified by the factors representing UFW, seasonal variations, hourly peak factors, etc. Those factors are usually specified separately in the input file of the computer model.
- As a check, the total demands in Years 6 and 30 correspond to the maximum consumption day demands from Table A2.6 (column $Q_{\text{max,day}}$). For UFW = 10 and 20%, respectively:

$$((1258.7 - 108.3) \times 1.265/0.90) \times 24 + 1300 \approx 40{,}106 \text{ m}^3/\text{d}.$$
$$((2109.8 - 108.3) \times 1.265/0.80) \times 24 + 1300 \approx 77{,}255 \text{ m}^3/\text{d}.$$

Table A2.7. Average nodal consumption.

Node	At year 6		At year 30		Factor of increase
	Q (l/s)	Q (m^3/h)	Q (l/s)	Q (m^3/h)	
01	30.09	108.3	30.09	108.3	1.000
02	7.15	25.7	13.12	47.2	1.834
03	18.79	67.6	31.00	111.6	1.650
04	18.43	66.3	34.96	125.9	1.897
05	16.91	60.9	34.28	123.4	2.027
06	9.13	32.9	18.83	67.8	2.062
07	16.77	60.4	26.97	97.1	1.608
08	5.42	19.5	10.77	38.8	1.986
09	16.93	60.9	33.62	121.0	1.986
10	17.30	62.3	32.32	116.4	1.869
11	16.66	60.0	32.39	116.6	1.944
12	23.05	83.0	40.59	146.1	1.761
13	12.73	45.8	17.90	64.4	1.405
14	16.75	60.3	24.85	89.5	1.484
15	19.37	69.7	28.81	103.7	1.488
16	15.94	57.4	22.64	81.5	1.420
17	21.45	77.2	32.31	116.3	1.506
18	21.36	76.9	38.17	137.4	1.787
19	13.21	47.6	25.04	90.2	1.895
20	4.10	14.8	7.77	28.0	1.895
21	3.76	13.5	7.12	25.6	1.895
22	7.43	26.7	14.08	50.7	1.895
23	10.59	38.1	18.27	65.8	1.725
24	6.31	22.7	10.15	36.5	1.608
Total	349.64	1258.7	586.05	2109.8	1.676

- The increase factor shown in the table indicates the ratio between the demands at the end and the beginning of the design period.

Conclusions:
- Due to different growth percentages in the various town districts, the pattern of population (demand) distribution will change throughout the design period. It can be observed from the table that the demands in Nodes 02, 04–06, 08–12 and 18–23 grow faster than in the rest of the network, shifting the demand concentration towards the south of the town. This fact should also be taken into consideration while deciding on the network layout.

Comments:
- Analysis of the demand distribution is an important element of the network design. Larger pipe diameters are normally laid in, or on the route to, the areas of higher demand.
- Phased development of the system should follow the demand development throughout the design period.

Reading:
- Chapter 4, Paragraph 4.3: 'Computer models as design tools'.

A2.3.3 *Network layout*

The network design began by defining preliminary routes of the secondary mains. What is important to realise at this stage is the obvious hydraulic correlation between the system parameters:
- demand distribution throughout the area,
- location and capacity of the supply points and their connection to the system,
- pipe diameters,
- pump units, their type, number and operating schedule,
- position, elevation and volume of the storage.

One single combination of the above parameters results in one unique distribution of the pressures and flows in the system. Modifying even a single nodal demand or pipe diameter will disturb this equilibrium, to some degree.

In theory, different network configurations can show a similar hydraulic performance provided that correct judgements have been made while building the model. For a final choice of the network configuration, other criteria such as: reliability assessment, soil conditions, access for maintenance, future extensions and etc. should also be considered.

Alternative A – Direct pumping
Based on the preliminary design concept and the demand analysis, the first model has been created for the demand situation after the commissioning

of the system (Year 6). The given EPANET filename is S6A1.NET and its contents can be studied from the enclosed CD. The main components of the system have been modelled as follows:

- [JUNCTIONS]: An average nodal consumption as shown in Table A2.7 in Year 6 has been introduced in each node.
- [JUNCTIONS]: Altitudes of nodes 102 and 202 that are the actual supply points have been assumed to equal the ground elevations taken from Figure A2.2.
- [DEMANDS]: The multiplier of the nodal demand has been set for the conditions on the maximum consumption day, including 10% UFW; the factor has been calculated as 1.265/(1–10/100) = 1.406.
- [RESERVOIRS]: The pump suction heads at the sources are assumed to be fixed at the ground elevation.
- [PIPES]: Given the fact that the population density is going to increase in the southern part of the town, the connection of the first source has been tested in Nodes 06 and 02. The pipe lengths are estimated based on the scale of Figure A2.2: for Pipe 102–06 at 1550 m and for 102–02 at 1700 m.
- [PIPES]: The factory is connected at Node 16 (pipe length 01–16 = 350 m). To provide good connectivity with Source 2, via the shortest path along route 16–17–18, this source will be connected in Node 18 (pipe length 202–18 = 750 m).
- [PUMPS]: Equal-size pumping stations have been assumed at the sources, with the following duty head and flow: $H_d = 50$ mwc; this is an estimate to satisfy the design pressure criteria, and $Q_d = 230$ l/s is an estimate based on the information from Table A2.6 (at Year 6: $Q_{max,day} = 40{,}106$ m^3/d ≈ 460 l/s). For the duty head and flow specified as a single pair of Q/H points, EPANET generates a synthetic pump curve based on the principle: Q_p (at $H_p = 0$) equals $2 \times Q_d$, and H_p (at $Q_p = 0$) equal to $1.33 \times H_d$. As a result, the following three Q/H points define the curve: 0–66.5, 230–50 and 460–0 (in l/s-mwc).

To analyse the demand distribution in the system, the first simulations have been run using uniform pipe diameters of 222 mm; the results are shown in Figures A2.5 and A2.6.

Clarification:

- The graphs show the snapshot at 12:00 hours (the maximum consumption hour).
- The pipe head-loss is displayed in m/km.
- With regard to the pumping stations, the negative figures indicate the actual pumping head in mwc.
- The reservoir symbols represent the suction nodes of the pumping stations.

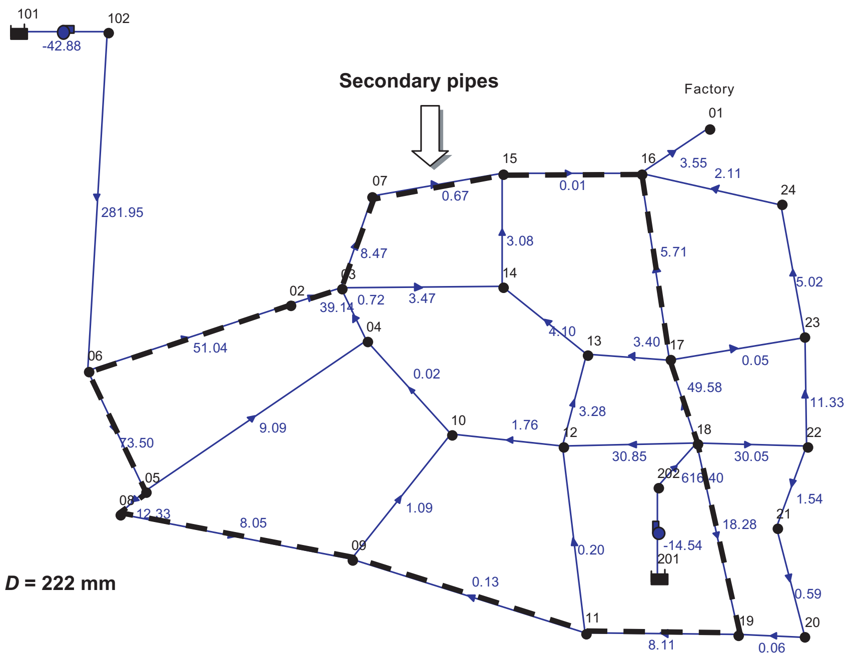


Figure A2.5. S6A1 (Source 1–06): head-loss at 12:00.

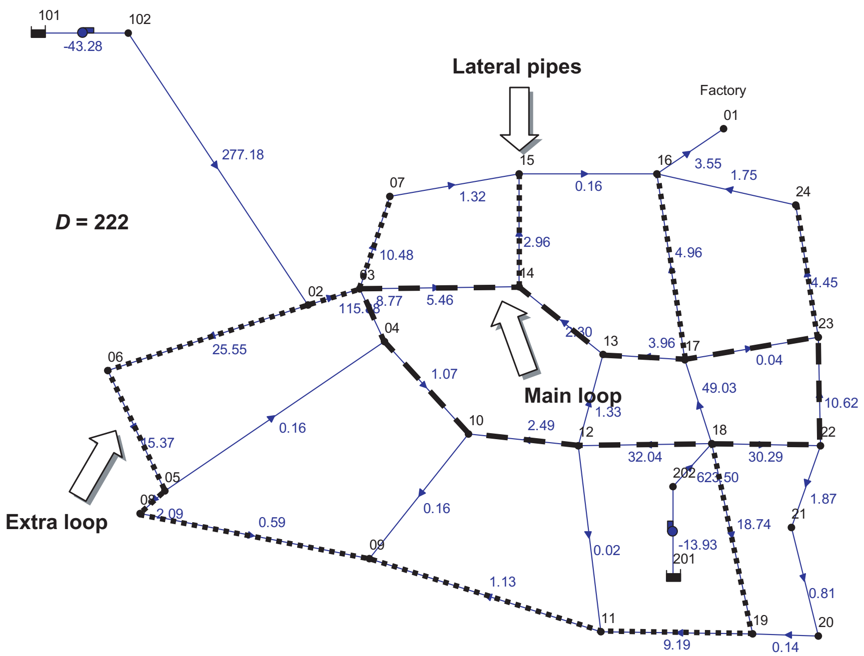


Figure A2.6. S6A1 (Source 1–02): head-loss at 12:00.

Conclusions:

- The source connection to different nodes influences the head-loss (flow) distribution.
- The pipes with a large head-loss (greater than 5–10 m/km) imply the routes of the bulk flows, which is a direct consequence of the demand distribution in the nodes. The diameter of these pipes has to be increased whereas the pipes with smaller head-loss should be reduced in size.
- Connecting Source 1 to Node 06 suggests that the secondary pipes should pass through the peripheral area of the town; a possible main loop is indicated in Figure A2.5. The logic behind it is to connect both sources and the factory with pipes that can carry additional capacity in case of accidents in the system.
- An alternative connection of Source 1 to Node 02 shifts the resistance towards the central part of the system; the main loop is shown in Figure A2.6. This loop is shorter (= cheaper) but may not provide a satisfactory distribution of water throughout the network. An extra loop or larger lateral pipes should therefore be considered.
- Out of the two equal pumping curves selected for the pumping stations, the one at Source 2 shows a lower head (i.e. higher supply), which is in contradiction to the initial requirement that both sources should supply a similar capacity. The likely cause is in the higher elevation of this pumping station compared to that in Source 1. Consequently, the pumping station at Source 2 can be smaller than the one at Source 1 (to be tested later).

In order to proceed, the layout from Figure A2.5 has been further developed:

- [PIPES]: The main loop from the figure has been formed from pipes $D = 400$ mm,
- [PIPES]: The source connections to the system were provided with pipes $D = 600$ mm,
- [PIPES]: All other pipes in the network have been set at $D = 200$ mm.

The input file has been named S6A2.NET, with simulation results shown in Figure A2.7.

Conclusions:

- The head-losses in most of the secondary pipes are low, suggesting that $D = 400$ mm is too large. Two possible solutions are
  - to look at each pipe individually and reduce the diameters where required.
  - to choose a smaller loop of secondary pipes.

The first approach in the conclusions actually means abandoning the loop concept and switching to a branched structure of the secondary

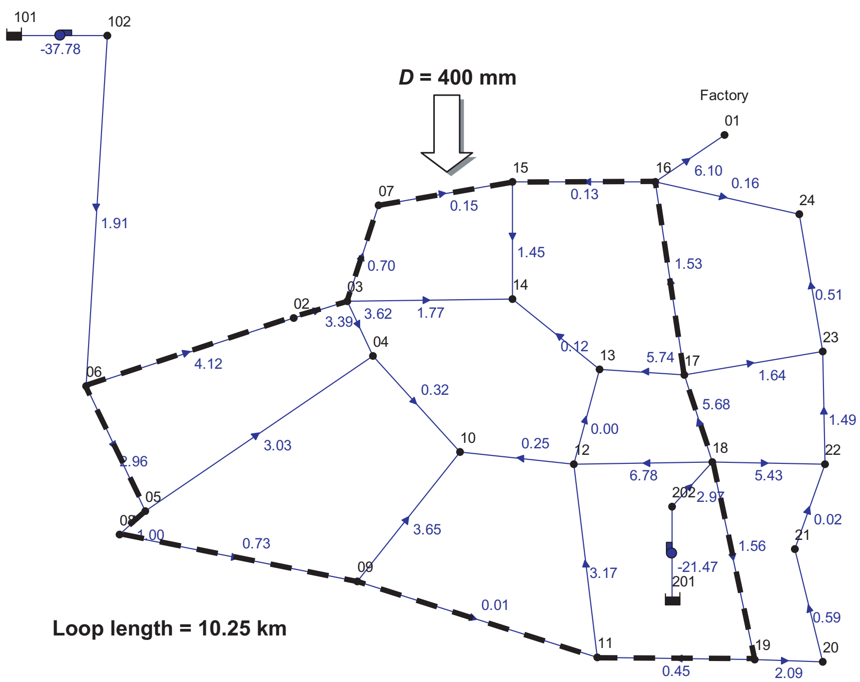


Figure A2.7. S6A2: head-loss at 12:00.

mains, which could potentially raise problems related to the reliability of supply. For those reasons, the approach with a smaller loop has been tried in the S6A3.NET file, created in the same way as S6A2.NET. The proposed layout is shown in Figure A2.8.

Conclusions:

- The length of the loop has been reduced with a slight improvement of the head-loss. However, some pipes are still too large (e.g. 04–10 or 13–17).
- The lateral pipes at the northern side of the town (03–07, 14–15, 17–16 and 23–24) show large head-losses. This is the consequence of making the main loop smaller. These pipes should be enlarged.

The smaller loop of the secondary pipes combined with laterals stretched towards the ends of the system appears to be a promising concept. To explore in addition the effects of the connection between Source 1 and Node 02, file S6A4.NET has been created from S6A3.NET:

- by introducing connection 102–02 instead of 102–06,
- by reducing further the length of the main loop.

The layout with the results is shown in Figure A2.9.

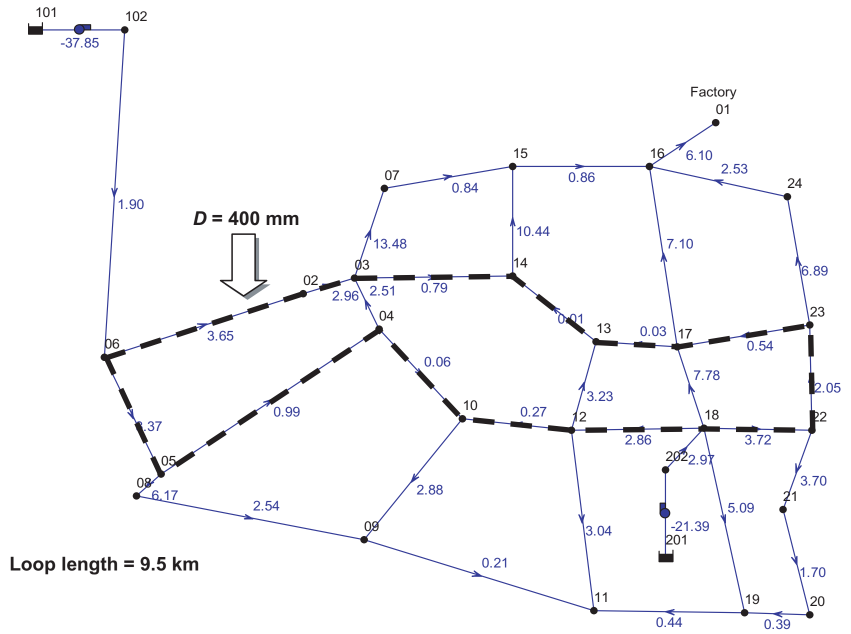


Figure A2.8. S6A3: head-loss at 12:00.

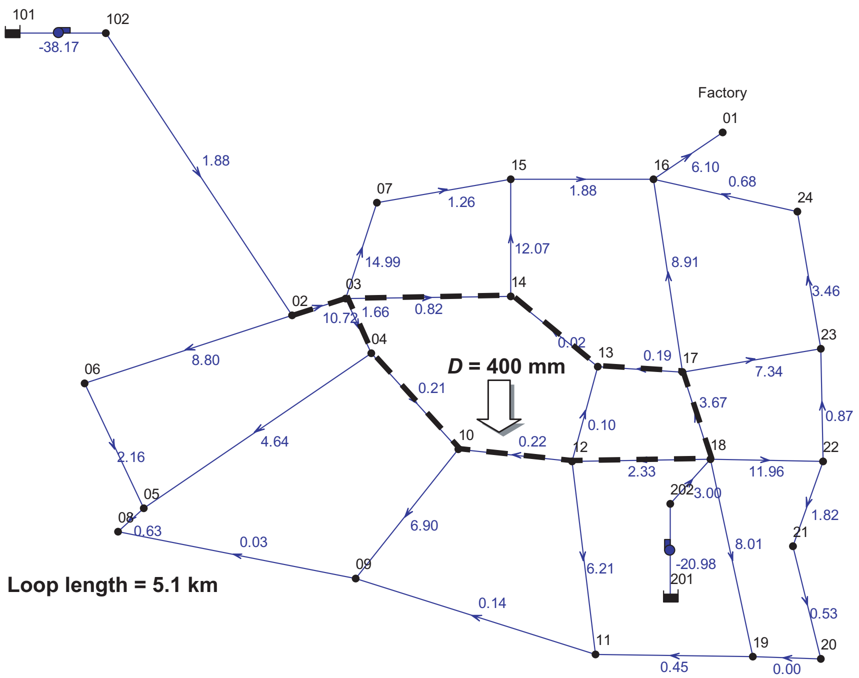


Figure A2.9. S6A4: head-loss at 12:00.

Conclusions:

- Connection 102–02 allows further reduction of the main loop. As a consequence, the head-losses in the lateral pipes will increase.
- The head-losses in the main loop remain low, which implies again that D = 400 mm is too large a diameter for the supply conditions at the beginning of the design period.

The network layout is further developed in file S6A5.INP. Through a number of trial and error computer runs, the pipe diameters are adopted to satisfy the design head-loss of ± 1–5 m/km. The final layout is shown in Figure A2.10.

To satisfy the minimum design pressure of 20 mwc while supplying a similar flow from both sources, the pumping characteristics have been adjusted as follows:

- Source 1: $H_{d,1}$ = 60 mwc, $Q_{d,1}$ = 270 l/s,
- Source 2: $H_{d,2}$ = 40 mwc, $Q_{d,2}$ = 250 l/s.

The results of calculation with these values are shown in Figures A2.11 and A2.12.

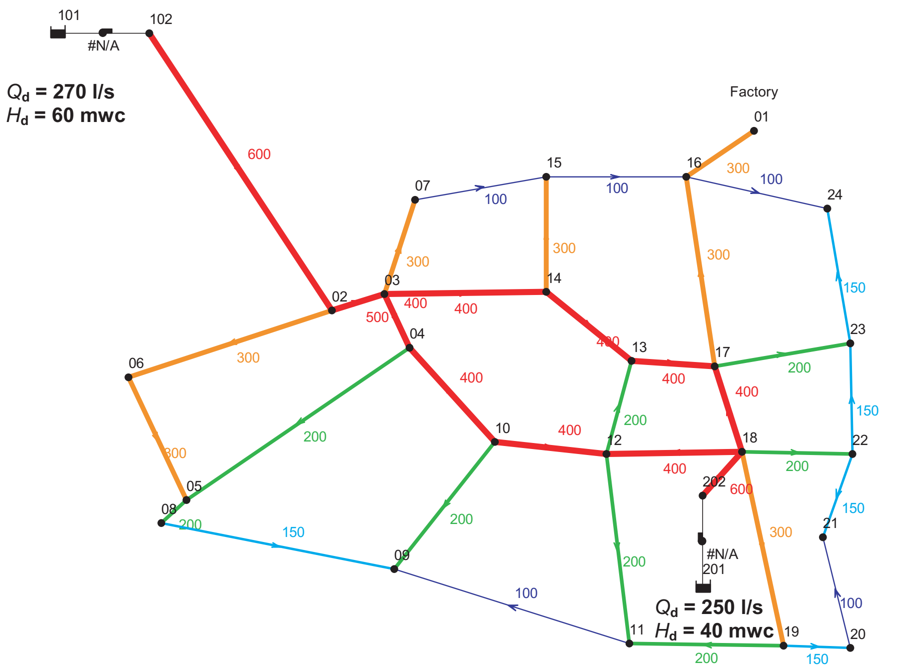


Figure A2.10. S6A5: pipe diameters.

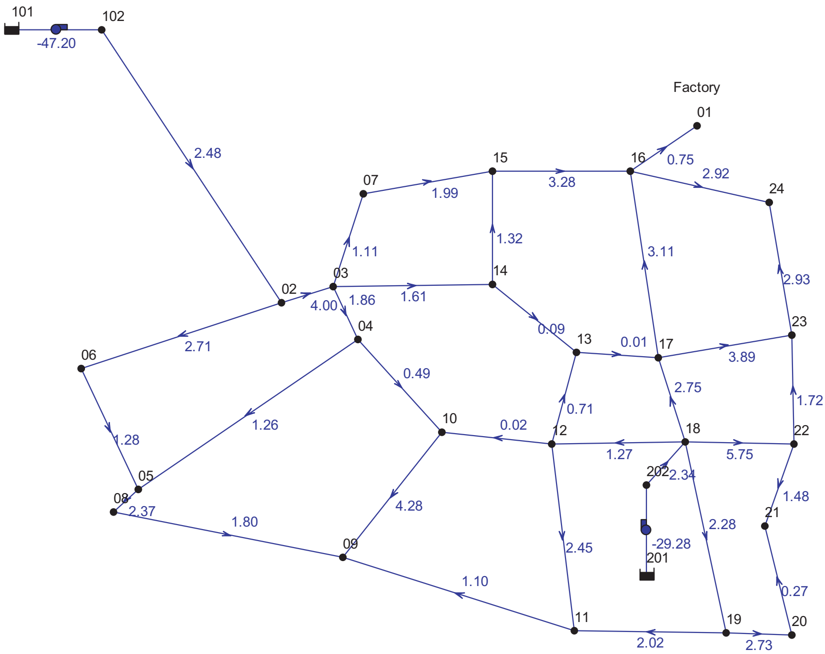


Figure A2.11. S6A5: head-loss at 12:00.

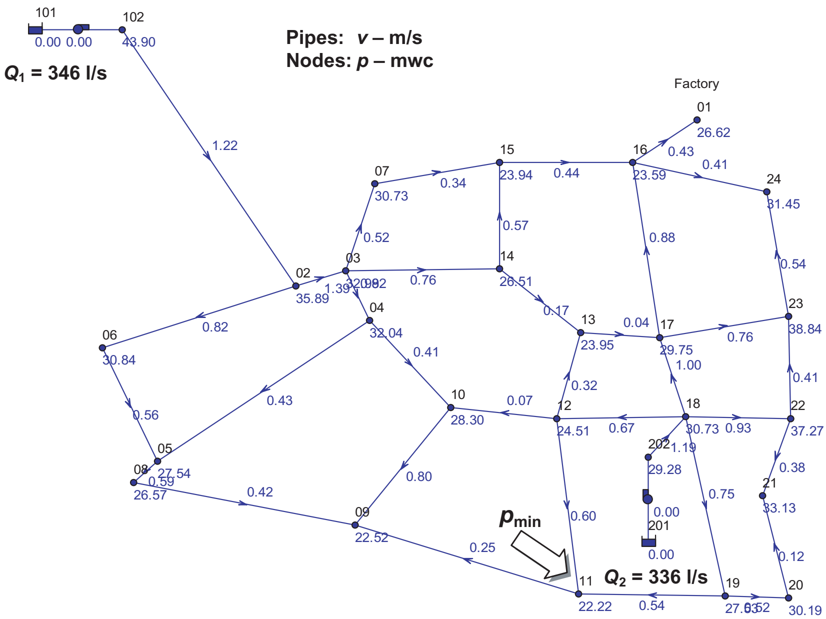


Figure A2.12. S6A5: velocities and pressures at 12:00.

Conclusions:

- As preferred, the pumping stations now work at an even capacity. The total supply (346 + 336 = 682 l/s = 2455 m^3/h) equals approximately the maximum hour on the maximum consumption day at the beginning of the design period (in Table A2.6, $Q_{\text{max,hour}} = 2452.9$ m^3/h at Year 6).
- The system has been deliberately left with extra capacity at this stage of the analysis. Lower velocities (head-losses) indicate that the pipes possess some spare capacity, which can be of use during irregular supply conditions. Second, the larger pipes can bear demand growth in the longer term, which postpones the system extension. Justification of these assumptions is going to be tested whilst analysing the system operation.
- The minimum pressure in the system is in Node 11 (22.22 mwc). The maximum (night) pressures are to be analysed after the number of pump units, their arrangement and operation has been decided.
- Assuming the unit costs as given in Table A2.3, the total investment cost for the pipes is calculated at US$4,661,500.

For reasons of comparison, two additional alternatives have been developed based on the design hydraulic gradient of ± 1–5 m/km and minimum pressure of 20 mwc:

- An alternative with the large loop of secondary mains, as discussed in Figure A2.5, is developed in file S6A6.NET. This network layout is shown in Figure A2.13. Similar hydraulic performance (Figure A2.14) yields the total cost of the pipes as US$5,218,500.
- A branched layout of the secondary mains (file S6A7.INP), shown in Figures A2.15 and A2.16.The total cost of the pipes in this case is US$4,484,500.

In both cases the same pump characteristics were used as in the S6A5 layout.

Conclusions:

- All three configurations, A5, A6 and A7, provide similar hydraulic performances during the maximum hour on the maximum consumption day. However, given the pattern of the population growth and reliability requirements, the S6A5 layout is anticipated to be the most attractive one. Further development of the direct pumping alternative is going to be based on this layout.
- The S6A6 layout is more expensive than the other two. Nevertheless, this is an acceptable approach in situations where the growth in population is followed by an overall growth of the town territory. As this is not anticipated in this case, additional investment in a large loop of secondary mains does not seem to be justified.

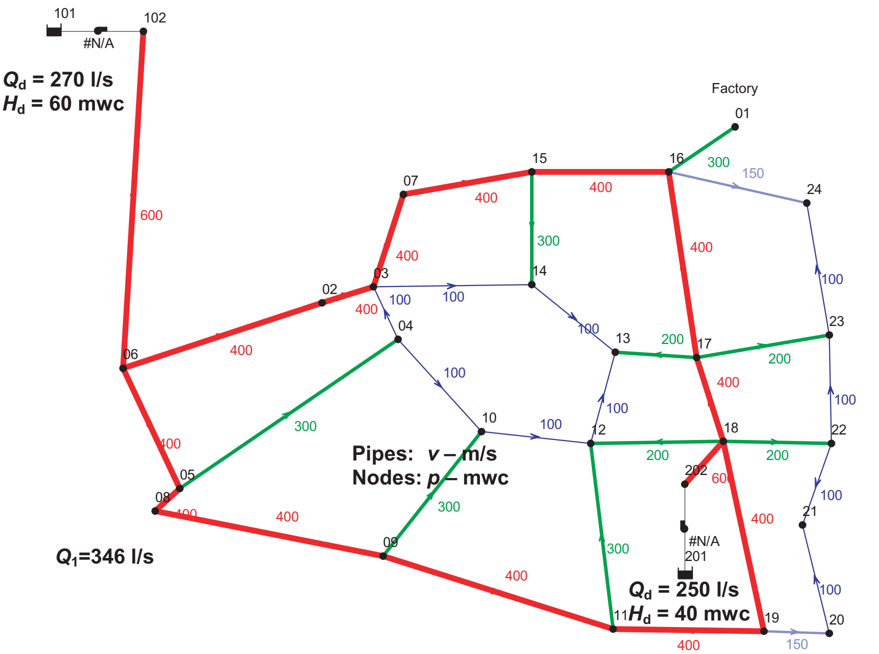


Figure A2.13. S6A6: pipe diameters for a large loop.

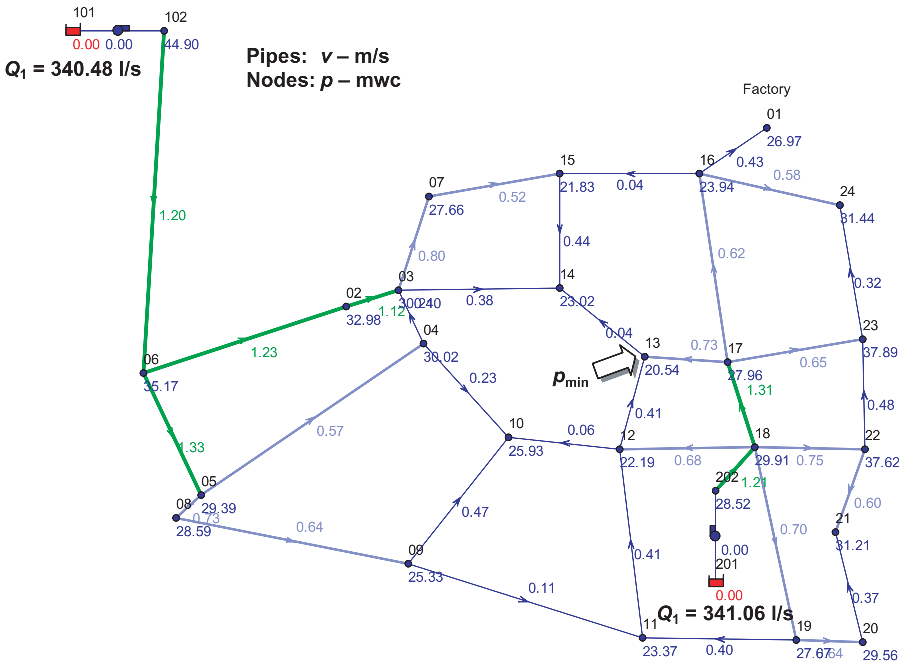


Figure A2.14. S6A6: velocities and pressures at 12:00.

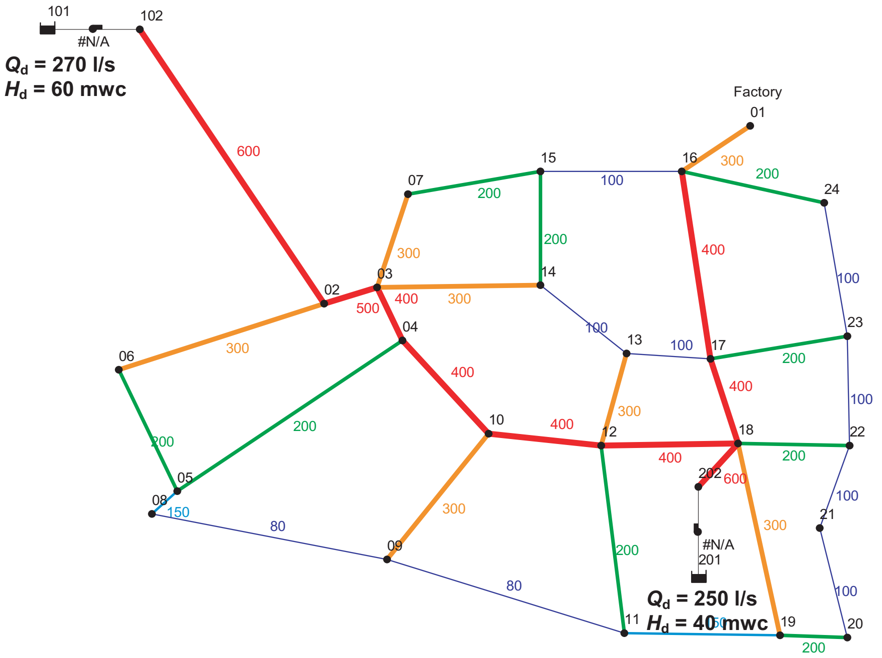


Figure A2.15. S6A7: branched mains – pipe diameters.

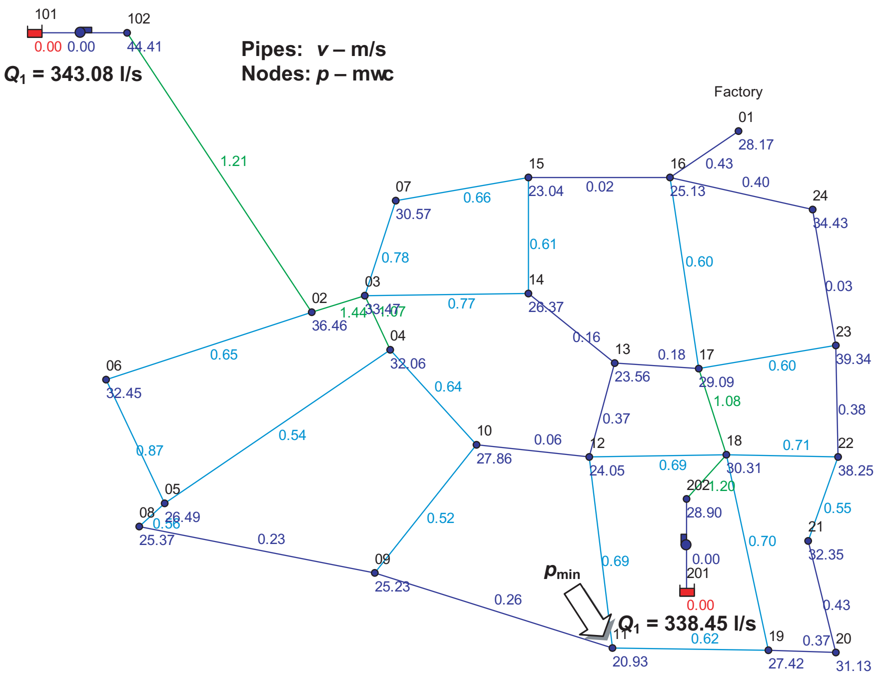


Figure A2.16. S6A7: Velocities and pressures at 12:00.

- The S6A7 layout is the cheapest but probably the least reliable. The cost saving compared to the S6A5 alternative does not itself make this layout more attractive. Its weak points will be illustrated during the reliability assessment of the network.

Comments:

- While sizing pipes:
  - A pipe with extremely low velocity/head-loss is a potential source of water quality problems and should therefore be reduced in diameter. Pipes in which the water stagnates play no role in a proper distribution network. These can be removed without major implications for the overall hydraulic performance of the network, unless needed for service connections or as a reliability provision.
  - Pipes with extremely high velocity/head-loss require high energy input for distribution and should therefore be increased in diameter. Often it is necessary to enlarge not just one but a sequence of pipes following the same path/flow direction (i.e. connected in series).

- Choosing appropriate pipe diameters means optimisation of the hydraulic gradients/flow velocities in the system. However, this does not mean:
  - satisfying the design hydraulic gradient/flow velocity *in each pipe* of the network at a particular moment,
  - satisfying the design hydraulic gradient/flow velocity of a single pipe *at any moment*. Depending on the demand pattern, one pipe may generate a range of velocities over 24 hours that will be rather wide; this is normal.
  - Optimisation of pipe diameters is usually done for the maximum consumption day demand. Thus, a pipe with extremely low velocities throughout that day will keep such velocities during the whole year. On the other hand, the velocities/head-losses tend to increase throughout the design period, as a result of the demand increase, and the same pipe may reach optimal operation within a couple of years.

To finalise the network design of Alternative A, the proposed layout in S6A5 has also been tested for the demand level at the end of the design period, i.e. 30 years from now. File S30A1.NET has been created from S6A5.NET by modifying the following sections:

- [JUNCTIONS]: The nodal consumption in Year 30 has been put in (Table A2.7).
- [DEMANDS]: The multiplier of the nodal consumption represents the maximum consumption day with UFW of 20%.

The simulation has been run with the same pump characteristics as at the beginning of the design period. The results are shown in Figure A2.17.

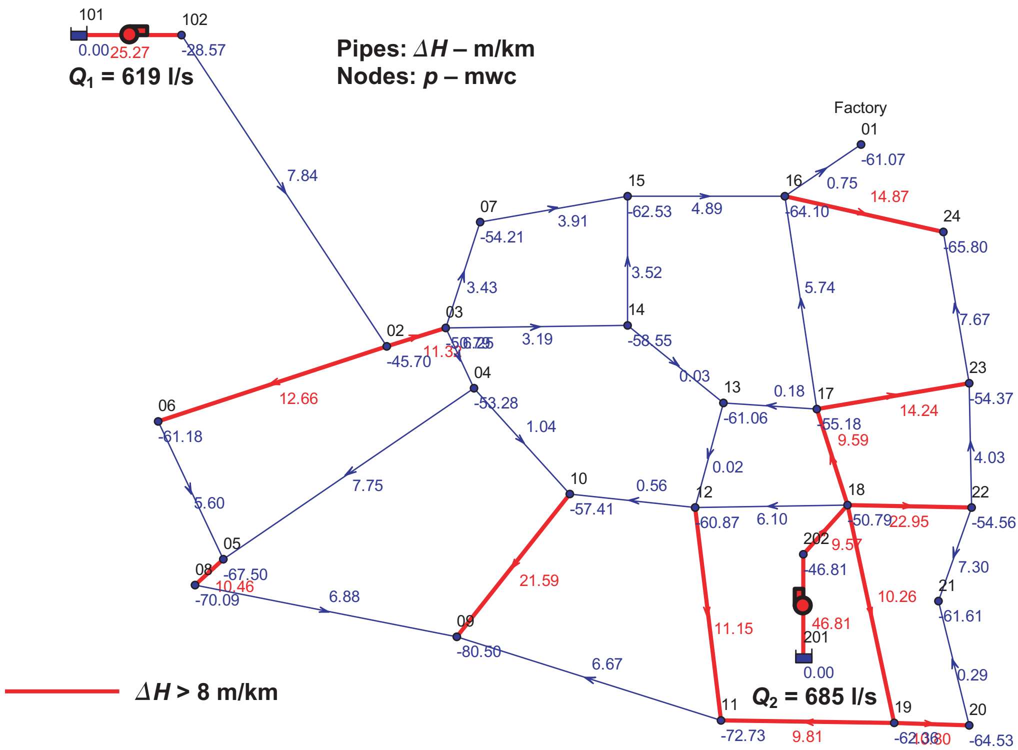


Figure A2.17. S30A1: head-losses and pressures at 12:00.

Conclusions:

- The nodal pressures become negative at a minimum of − 80.54 mwc at Node 09. This is a consequence of the demand/population growth that has created two problems:
  - the existing pumps are too weak to deliver the increased flow at sufficient head (the pressures become negative already at the nodes nearest to the supply points),
  - the flow increase causes a substantial increase of the head-loss in some pipes (ΔH> 8 m/km).
- The warning message in EPANET has also already indicated too weak pumping, which was registered in 12 out of 24 hours; the system has an insufficient supply for at least half of the day.
- Between midnight and 06:00 hours in the morning, the system is able to satisfy demand at the required minimum pressure. The problems occur from 07:00 hours onwards. At that moment the pressure will become negative for the first time, while the head-loss in some pipes will start to grow, as Figure A2.18 shows. The total demand of the system at 07:00 is 979 l/s ≈ 3526 m^3/h, which is the level of demand

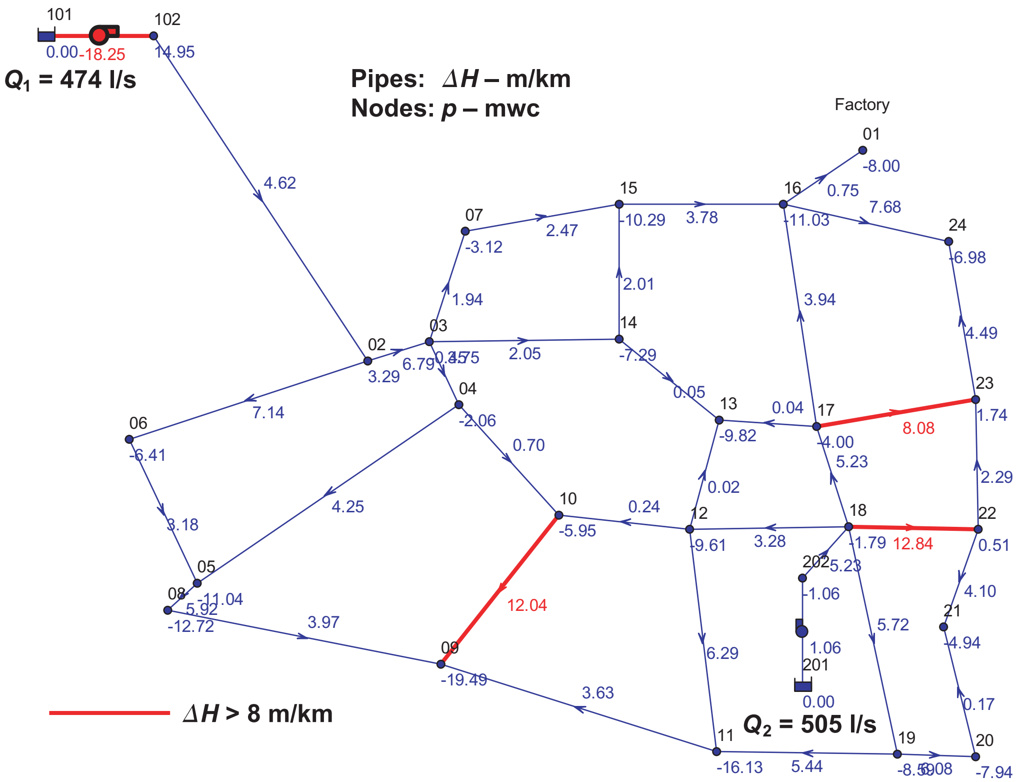


Figure A2.18. S30A1: head-losses and pressures at 07:00.

at the maximum hour of the maximum consumption day in around the Year 20–21 (see column $Q_{max,hour}$ in Table A2.6). This is the critical moment but the new investment is likely to be necessary a few years earlier, to maintain regular supply throughout the entire design period. These include:

- extension of the pumping stations (the duty flows have to be nearly doubled, based on the calculated demand increase during the design period),
- replacement of the critical pipes with larger ones or laying the pipes in parallel.

During the second phase, the pipes already laid will be approximately10–15 years old. These should normally still be in good condition and their replacement with a larger diameter pipe may not be economically justified. If reliability is an issue, parallel pipes could be considered instead but the trench widening is required in this case. In this stage, the combination of both measures has been applied as an illustration, for purely educational reasons.

Summary of Alternative A
The layouts for the regular supply conditions have been finalised in files S6A8.NET and S30A2.NET (beginning and end of the design period, respectively). The results are displayed in Figures A2.19–A2.22.

Clarification:

- The bold lines in Figure A2.20 indicate routes where action will be taken in around Year 15. Parallel pipes D = 500 mm are planned along routes 102–02, 02–03 and 202–18, while all other indicated pipes will be replaced with the next larger diameter from Table A2.3.
- At the beginning of the design period, the diameters of Pipes 102–02 and 202–18 will be reduced to D = 500 mm instead of 600 mm, as previously determined in S6A5. As a consequence, the pressure will drop by a few mwc compared to the values in Figure A2.12. To prevent

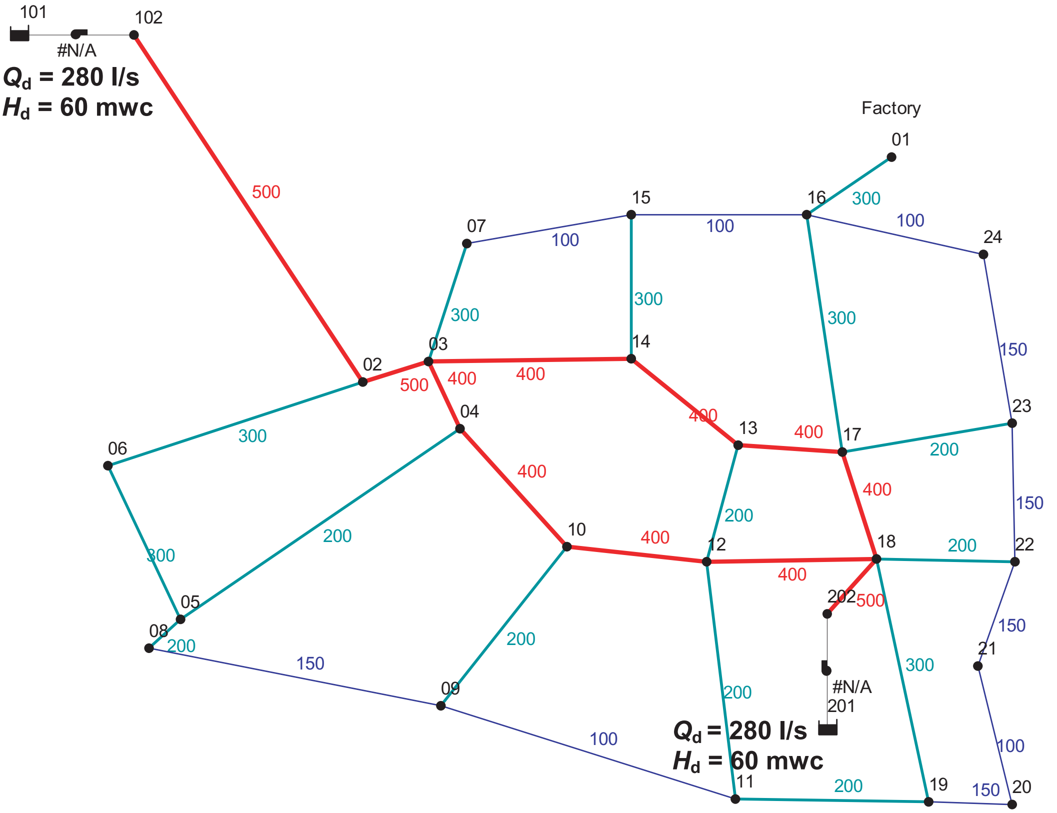


Figure A2.19. Alternative A: pipe diameters at the beginning of the design period.

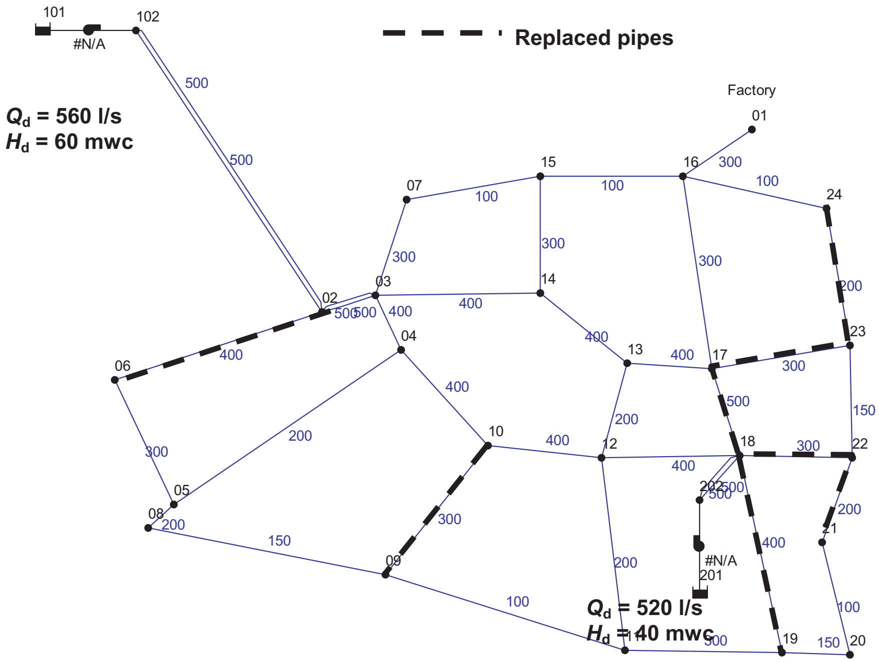


Figure A2.20. Alternative A: pipe diameters at the end of the design period.

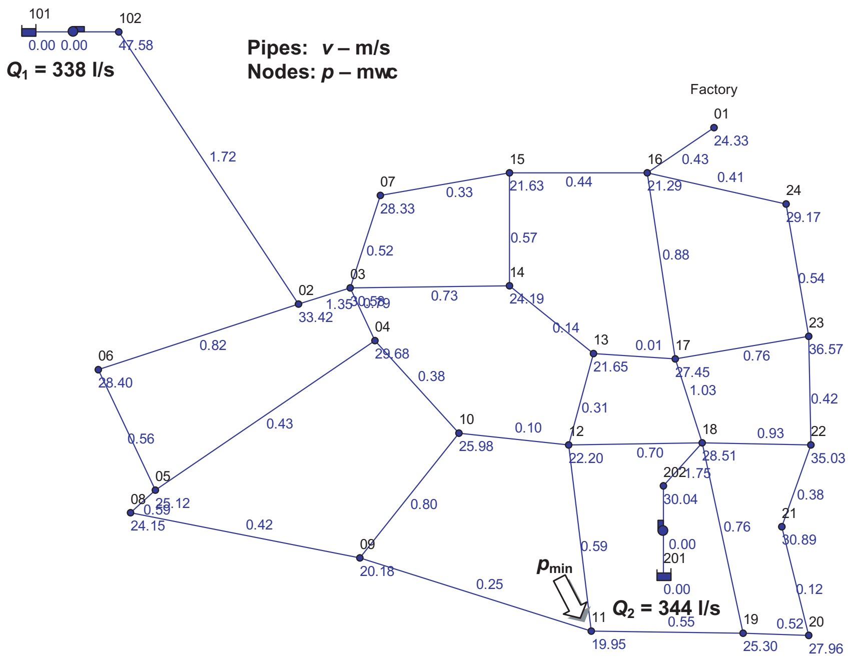


Figure A2.21. Alternative A: operation at 12:00 at the beginning of the design period.

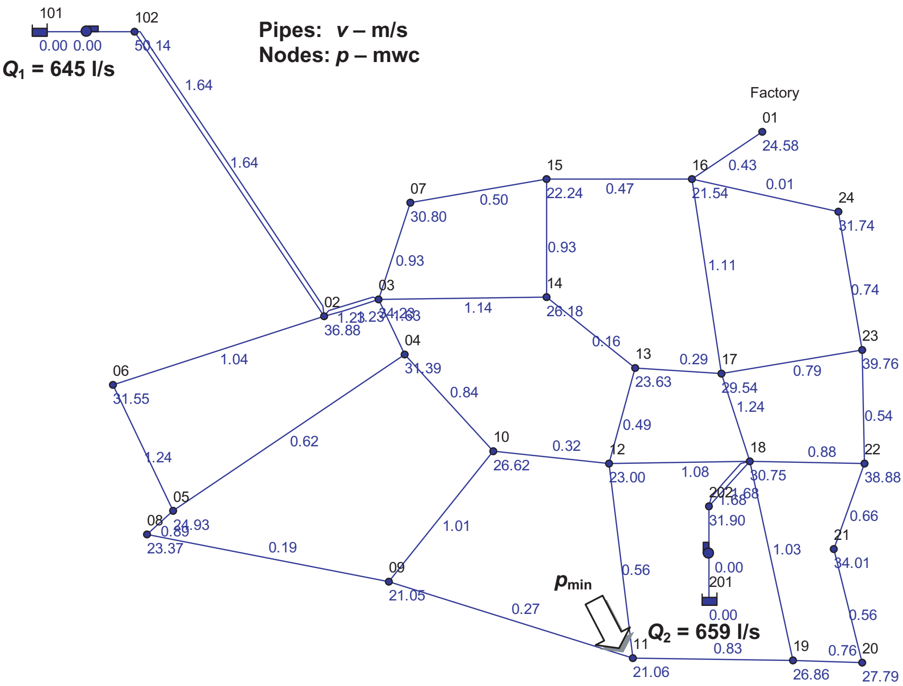


Figure A2.22. Alternative A: operation at 12:00 at the end of the design period.

it falling below $p_{min} \approx 20$ mwc, the pumping capacity has to be slightly increased. In Source 1, $Q_d = 280$ l/s (from 270 l/s), and in Source 2, $Q_d = 260$ l/s (from 250 l/s).

Conclusions:

- Pipe investment for the network at the beginning of the design period is US$4,539,000. Additional cost for the pipes laid in the second phase is US$2,330,000.
- Both network configurations provide stable operation during the regular (maximum) supply conditions. At the end of the design period, the total peak supply of the pumping stations fits the figure from Table A2.6 ($645 + 660 = 1305$ l/s $= 4698$ m^3/h $\approx Q_{max,\ hour}$ in Year 30). The first source will have reached 93% of its maximum capacity (2500 m^3/h = 695 l/s).
- For some pipes, there is still a room for further reduction of the diameter. Relatively low velocities are registered in Pipes 08–09, 09–11, 10–12, 14–13, 13–17, 15–16 and 16–24, some of those indicating a 'zero-line' between the part of the network supplied by Source 1, and the one supplied by Source 2. The final decision on this is going to be taken after the reliability aspects have been analysed.

Alternative B – Pumping and balancing storage
Based on the preliminary concept and applying the same basic principles, the network layout has been developed for the beginning (file S6B1.NET) and the end of the design period (file S30B1.NET). To utilise the existing topography, the tank has been positioned at the highest altitude in the area (28.0 mwc), in the vicinity of Node 13. The layouts and the results of the simulation are shown in Figures A2.23–A2.28.

Clarification:

- As the first approximation, the tank surface level has been assumed constant throughout the entire period of simulation (the 'fixed head' node, which is modelled in EPANET as a reservoir). This gives an initial impression about the pressure distribution in the system. Two possibilities have been analysed:
  - a ground tank with a depth of 6 m (the fixed head = 28 + 6 = 34 msl),
  - an elevated tank ± 20 m high including ± 2 m water depth (the fixed head = 28 + 22 = 50 msl).
- Based on the previous experience, the pump characteristics have been set as follows:
  - beginning of the design period: $H_{d,1}$ = 60 mwc, $Q_{d,1}$ = 460 l/s = Q_{maxd} at Year 6,

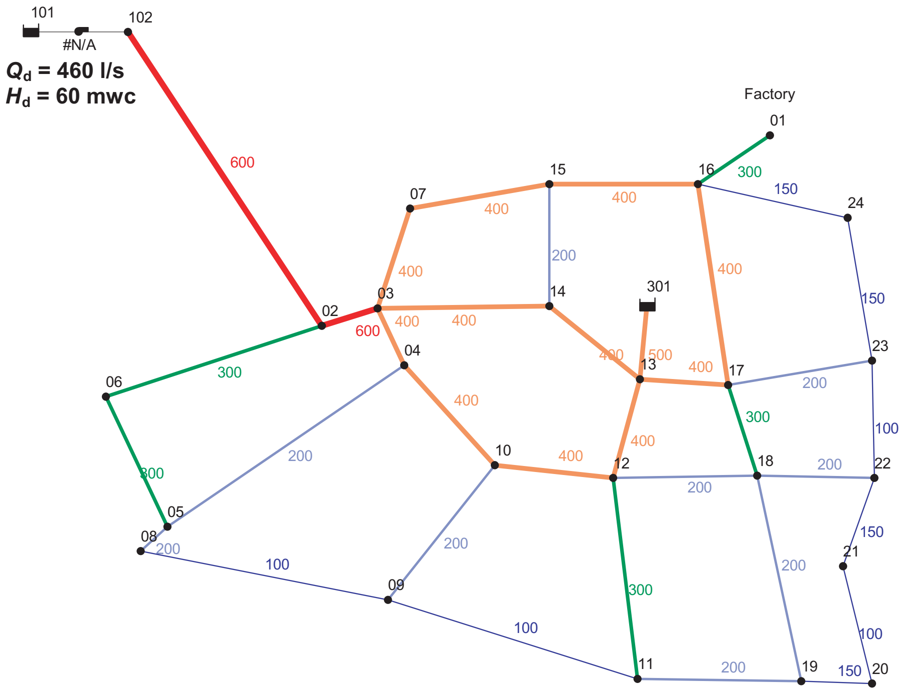


Figure A2.23. Alternative B: pipe diameters at the beginning of the design period.

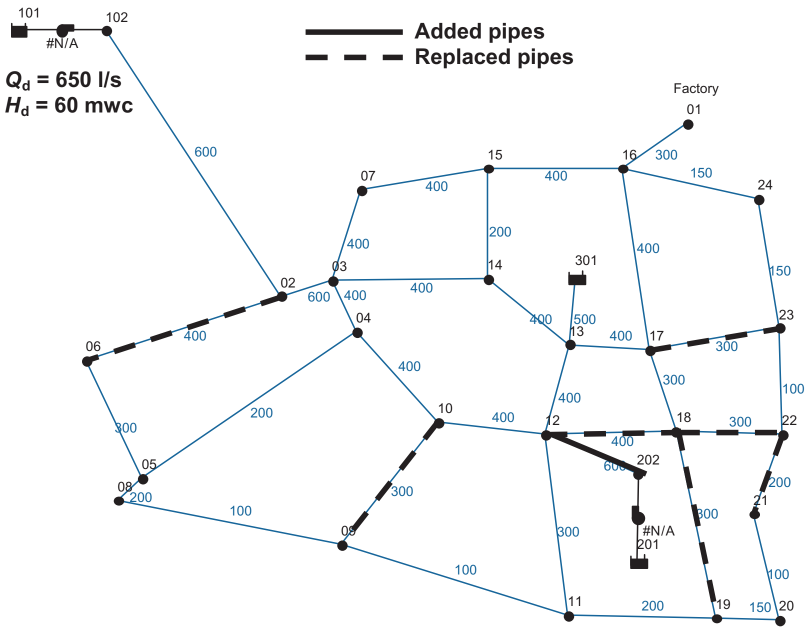


Figure A2.24. Alternative B: pipe diameters at the end of the design period.

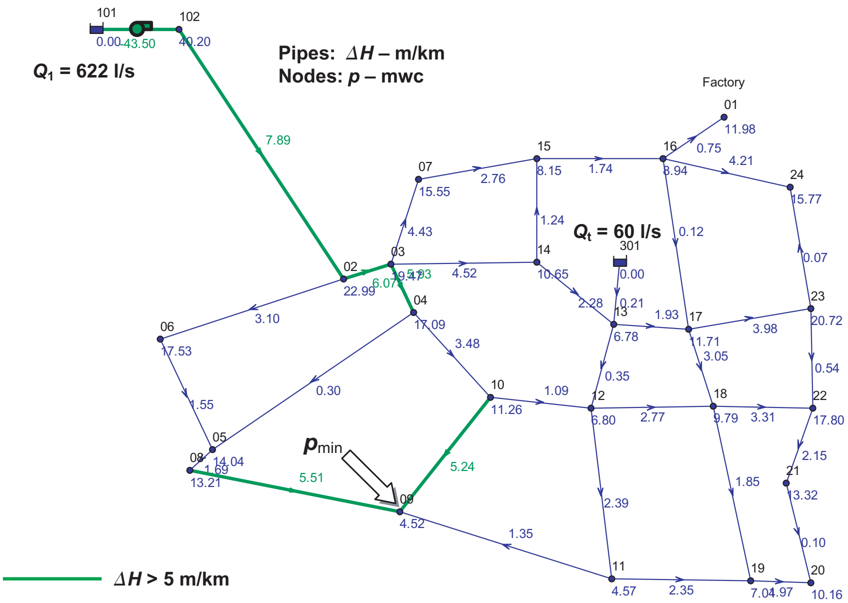


Figure A2.25. Alternative B: ground tank, operation at 12:00 at the beginning of the design period.

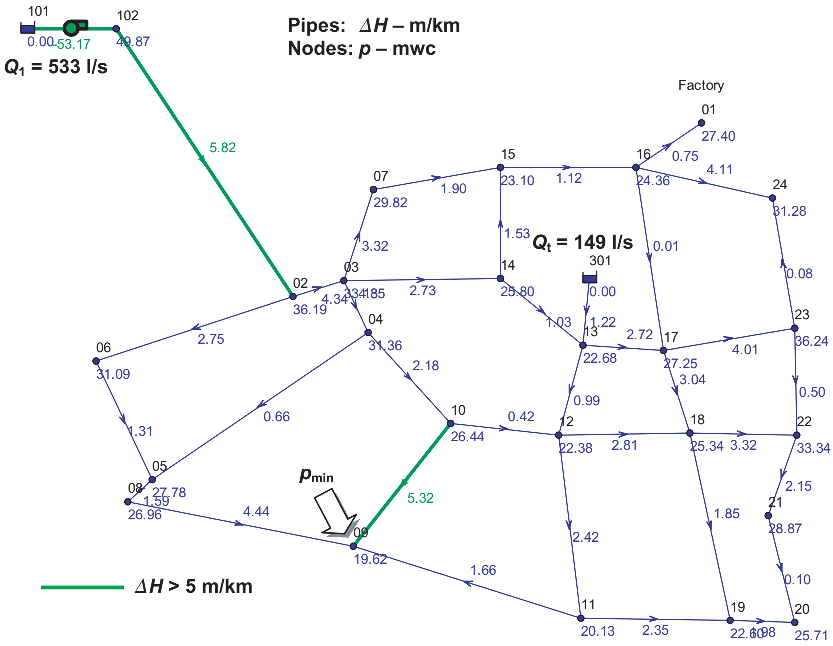


Figure A2.26. Alternative B: elevated tank, operation at 12:00 at the beginning of the design period.

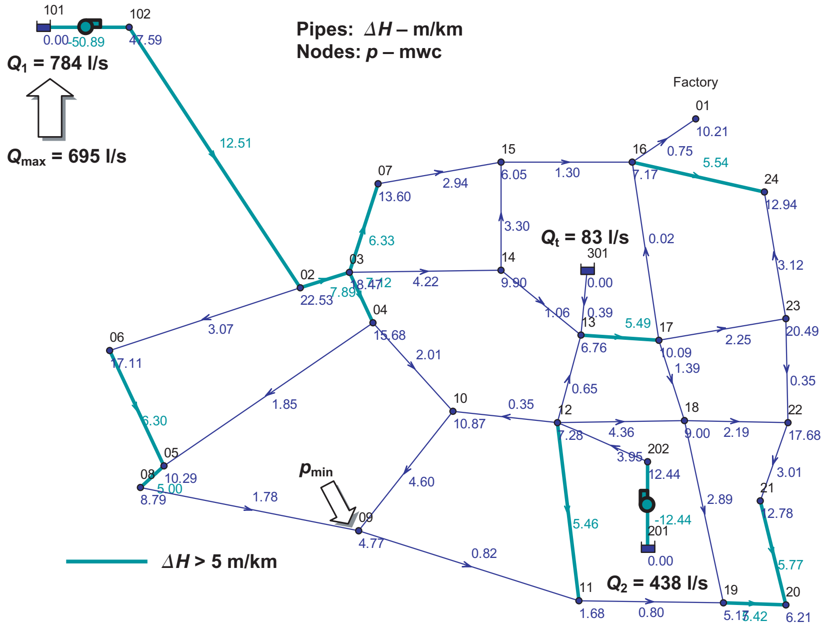


Figure A2.27. Alternative B: ground tank, operation at 12:00 at the end of the design period.

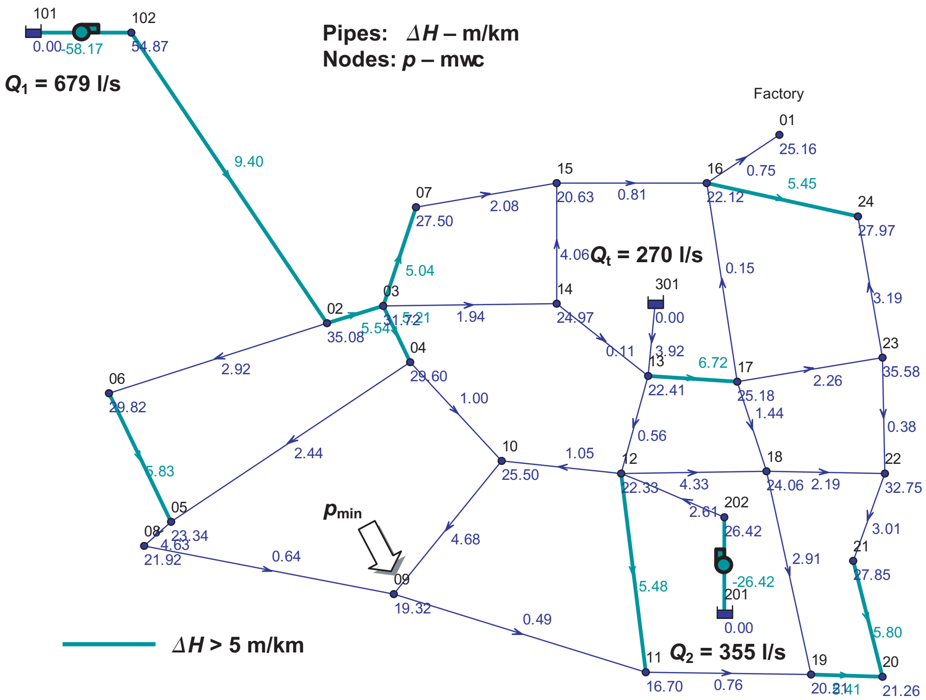


Figure A2.28. Alternative B: elevated tank, operation at 12:00 at the end of the design period.

- end of the design period: $H_{d,1} = 60$ mwc, $Q_{d,1} = 650$ l/s ($\approx Q_{max}$ of Source 1), and $H_{d,2} = 40$ mwc, $Q_{d,2} = 250$ l/s ($Q_{d,1} + Q_{d,2} = 900$ l/s $\approx Q_{maxd}$ at Year 30).

- To provide a link with the main loop, the second source has been connected at Node 12.

Conclusions:

- Pipe investment for the network at the beginning of the design period is US$4,927,500. The additional cost for the pipes to be laid in approximately Year 20 is US$1,455,000. Compared to Alternative A, the saving is mainly obtained by avoiding parallel pipes from the sources.
- There is a difference in the head-loss distribution, depending on the level of the tank. It results from different quantities supplied from the tank and the pumping station(s). However, with a few exceptions, this does not have significant implications on the choice of pipe diameters.
- The ground tank draws too much water from the sources throughout the day. As a consequence, the required supply from Source 1 at the end of the design period exceeds its maximum capacity, which is 695 l/s. In addition, the nodal pressures in the system are too low.

- The elevated tank is showing much better hydraulic performance; the pumping is reduced due to increased water flow supplied from the tank. With the same pump characteristics, the pressures in the system are almost correct. However, erecting tanks with a large volume can be quite an expensive solution. The final decision on the tank height will be made after the balancing volume has been determined.

Comments:

- A balancing tank that is positioned too low receives more water than it can supply to the system. Such a tank becomes full after some time. To prevent this, the pumping at the source can be reduced, which will slow down the filling of the tank. Another possibility would be to install an additional pumping station in the vicinity of the tank, which will empty its volume, thereby providing sufficient pressure.
- A balancing tank that is positioned too high will receive less water than it should supply to the system and therefore will soon become empty. To prevent this, the pumping head at the source should be increased.
- The amount of water entering and leaving the tank are known in advance, if the supply source has been tuned to provide the average flow throughout the day. A sophisticated way to quickly evaluate the tank position is to model it as an ordinary node with the balancing flows as its consumption. Such consumption would fluctuate between the maximum or minimum hour consumption reduced by the average flow supplied by the pump. It becomes negative when the tank is supplying the system and positive when its volume is being recovered. The pressure in the node, resulting from the simulations for two extreme consumptions, can give an impression about the range of water depths in the tank; it should therefore be reasonably high and within a range of a few metres.

A2.3.4 *Pumping heads and flows*

The pumping heads and flows determined at the beginning and the end of the design period are summarised in Tables A2.8 and A2.9. As expected, more pumping will be involved in Alternative A than in the case of B.

Table A2.8. Alternative A: pumping stations.

	Beginning of the design period			End of the design period		
	Q_d (l/s)	H_d (mwc)	$Q_{max,hour}$ (l/s)	Q_d (l/s)	H_d (mwc)	$Q_{max,hour}$ (l/s)
Source 1	280	60	338	560	60	645
Source 2	260	40	344	520	40	660

Table A2.9. Alternative B: pumping stations.

	Beginning of the design period				End of the design period			
	Q_d (l/s)	H_d (mwc)	$Q_{max,hour}$ (l/s) Ground	$Q_{max,hour}$ (l/s) Elevated	Q_d (l/s)	H_d (mwc)	$Q_{max,hour}$ (l/s) Ground	$Q_{max,hour}$ (l/s) Elevated
Source 1	460	60	622	533	650	60	784	679
Source 2	—	—	—	—	250	40	438	355
Tank	—	—	60	149	—	—	83	271

Table A2.10. Preliminary pump selection.

	Alternative A				Alternative B (Elevated tank)			
	Q_p (l/s)	H_p (mwc)	No. of units begin	No. of units end	Q_p (l/s)	H_p (mwc)	No. of units begin	No. of units end
Source 1	100	60	3 + 1[1]	6 + 1	150	60	3 + 1	6 + 1
Source 2	90	40	3 + 1	6 + 1	130	40	—	2 + 1

[1] One unit is on stand-by.

In order to facilitate the demand variations over a period of 24 hours, the logical choice is to arrange the pumps in parallel. One possibility is shown in Table A2.10. The final choice of the pump type and number of units will take place after the network operation has been analysed.

Clarification:

- A hydraulically equivalent performance is reached between one pump of a particular Q_d and H_d, and n pumps of Q_d/n and H_d connected in parallel. For example, three units of 100 l/s against duty head of 60 mwc equal one unit of 300 l/s and the same duty head.

Application of booster stations

To illustrate the difficulties with pressure boosting in looped systems, file S6B2.NET has been created from S6B1.NET. The operation of the booster station has been tested in three different locations: from Nodes 13 to 17, 13–12 and 14–301 (instead of Pipe 14–13). In all cases the same duty head and flow have been used: $H_{d,bs} = 30$ mwc, $Q_{d,bs} = 270$ l/s. The results of calculation are shown in Figures A2.29–A2.33.

Conclusions:

- Putting a booster station in the system causes a pressure drop in nodes at the suction side of the pump.
- The booster station tends to re-circulate the water along the loop to which it belongs (loop 12–13–17–18 in Figure A2.29 and the same loop together with 14–13–12–10–04–03 in Figure A2.31). This

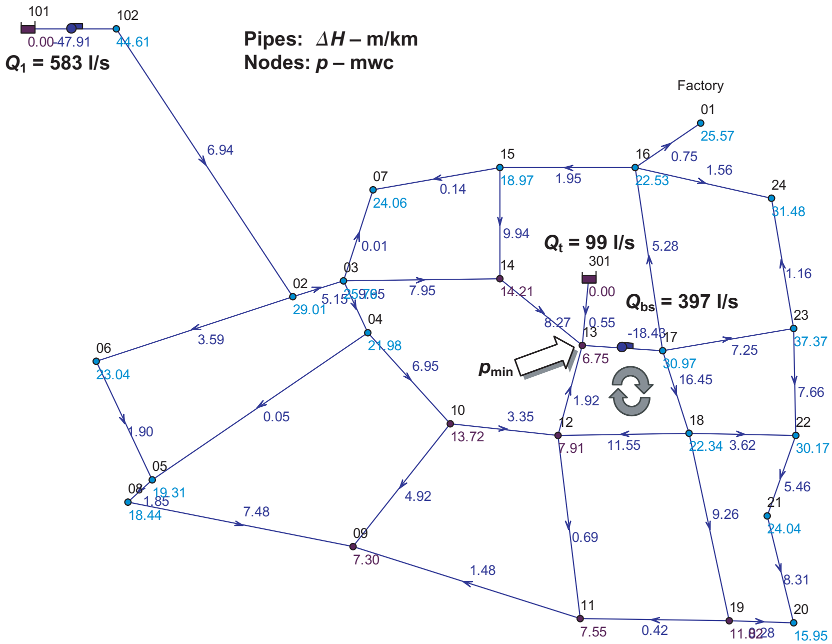


Figure A2.29. S6B2: ground tank and booster 13–17, operation at 12:00.

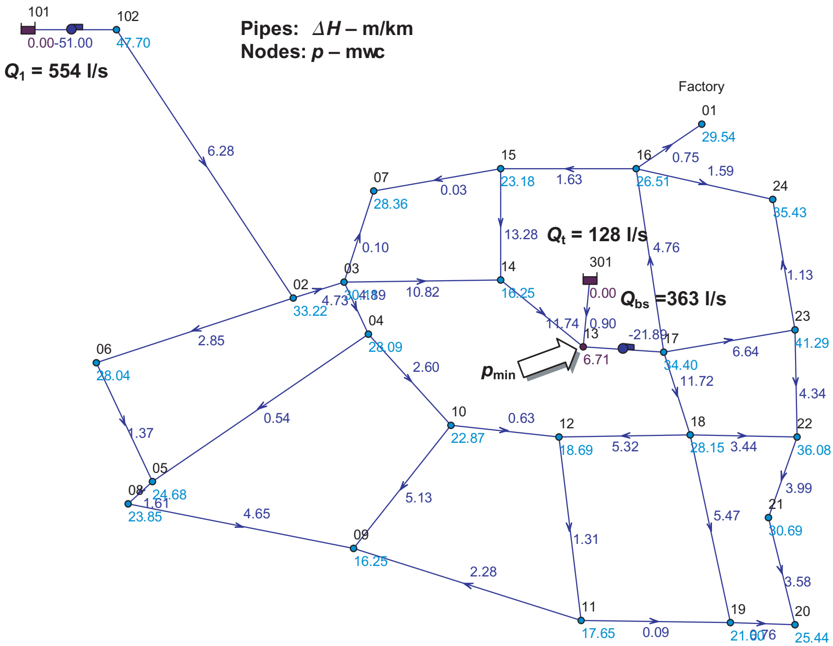


Figure A2.30. S6B2: booster 13–17, pipe 12–13 removed, at 12:00.

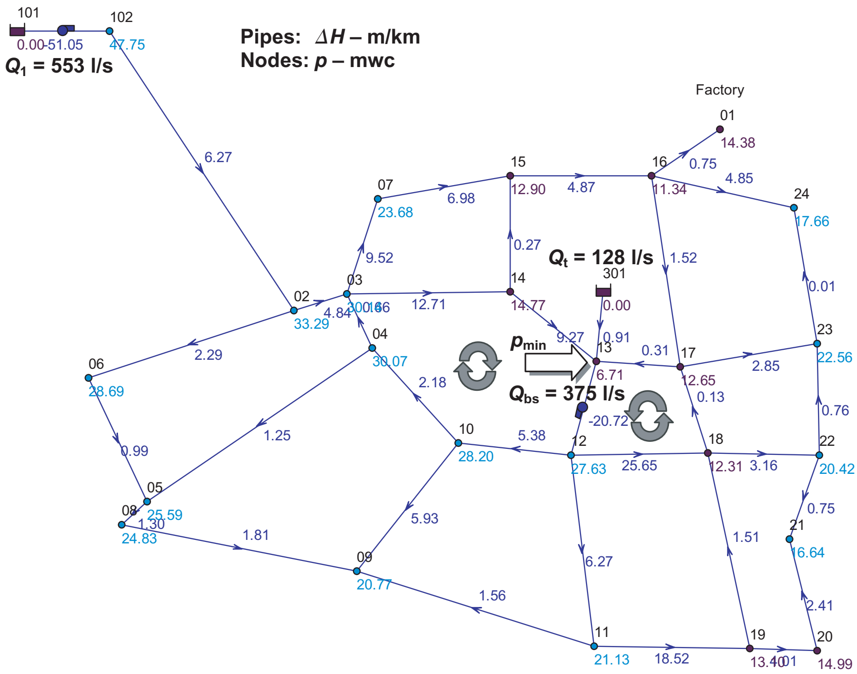


Figure A2.31. S6B2: ground tank and booster 13–12, at 12:00.

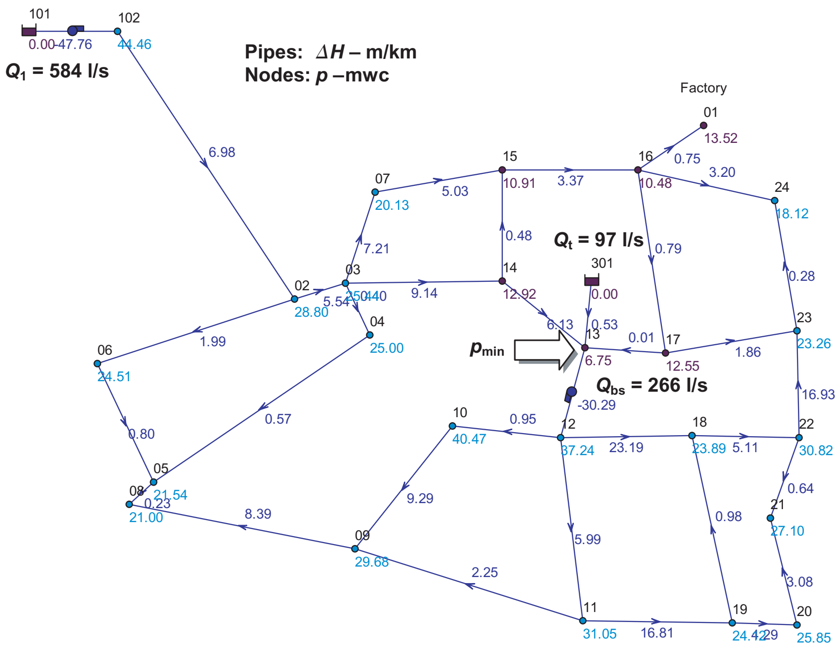


Figure A2.32. S6B2: booster 13–12, pipes 04–10 and 17–18 removed.

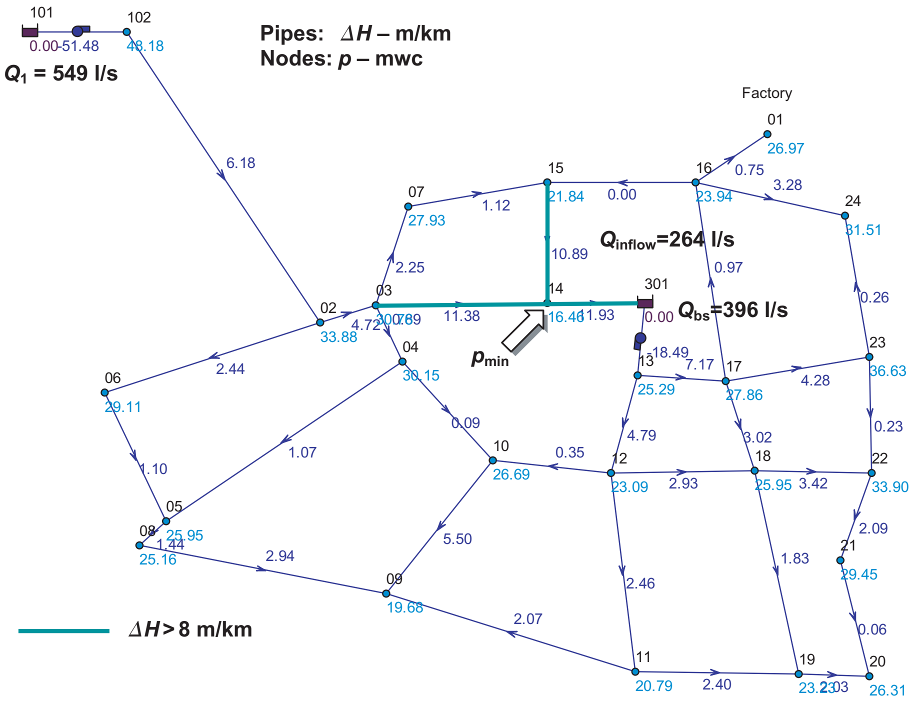


Figure A2.33. S6B2: ground tank and booster, at 12:00.

becomes obvious if one pipe is removed from the loop(s); the pressure downstream of the booster station will increase (Figures A2.30 and A2.32).

- The pipe resistance increases in the vicinity of the booster station. Contrary to expectations, enlarging the diameter of these pipes makes no sense as it creates negative effects on the pressure in the system. The fact is that, with larger pipes, even more water will be pushed towards the booster station, which reduces the pumping head.

To analyse the relation between the booster pumping capacity and the pipe resistance, the pipes indicated in Figure 33 have been replaced with the next larger diameter from Table A2.3 (file S6B3.NET). As a result (shown in Figure A2.34), the booster station increases the flow on account of the head (pressure). Installing much stronger pumps will reinstate the pressure, but will also initiate the old problem of water re-circulation and large pressure drops in some of the pipes. Operation of the system with doubled duty flow ($H_{d,bs}$ = 30 mwc, $Q_{d,bs}$ = 540 l/s) of the booster station has been shown in Figure A2.35.

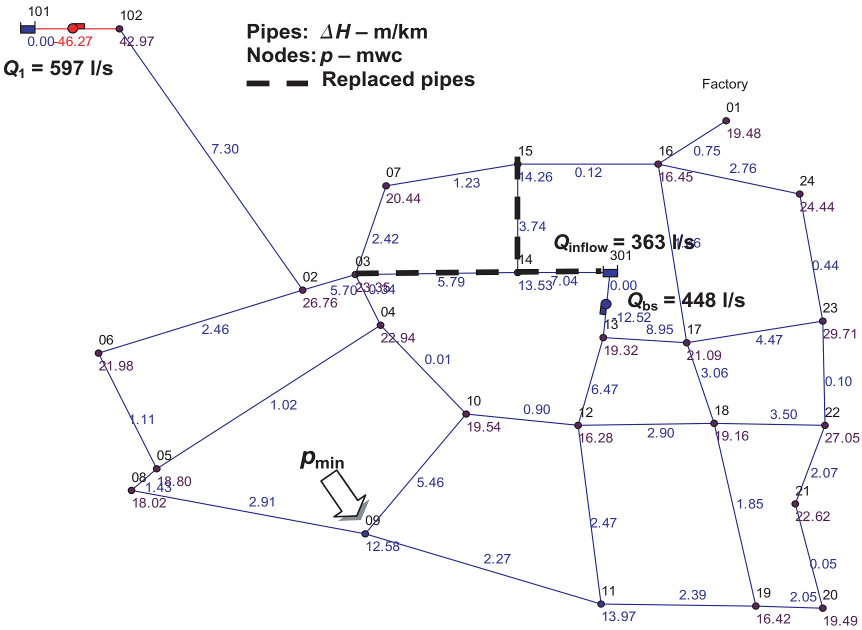


Figure A2.34. S6B3: pipe enlargement, operation at 12:00.

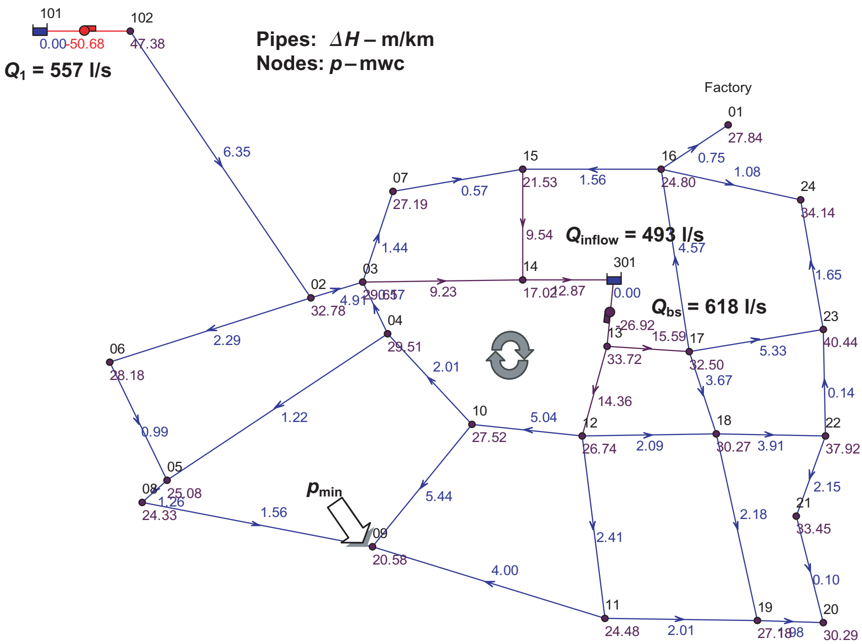


Figure A2.35. S6B3: booster pump enlargement, operation at 12:00.

Comments:

- Booster stations are useful if pressure has to be provided for higher zones of a distribution system, in which case they will be installed on a single line feeding a particular zone. They are of virtually no hydraulic use in applications within looped networks.
- Two common observations regarding the modelling of booster stations in EPANET are:
  - Compared to the modelling of a source pumping station, the suction node of a booster station can be any node within the network. The flow direction in the pump will be from the first to the second node selected on the map after clicking the pump button. The reversed flow is prevented by the programme (it shows $Q_{bs} = 0$).
  - A pipe replaced by a booster station has to be deleted first; otherwise the programme assumes two pipes in parallel where in fact one is with the booster pump.

Due to all the listed deficiencies, Alternative B with a ground tank and booster station is therefore discounted with regard to the Safi network.

A2.3.5 *Storage volume*

Based on the diurnal domestic consumption pattern from Figure A2.4 and the UFW levels as assumed in Table A2.6, the balancing volume at the beginning of the design period is calculated according to the principles explained in Section 4.2.3: 'Storage design' in Chapter 4. The results of a spreadsheet calculation are shown in Table A2.11.

Clarification:

- The spreadsheet calculation has been conducted for the maximum consumption day conditions. The total balancing volume equals $3833.7 + 539.2 = 4372.9 \approx 4400$ m^3.
- The same calculation for the demand at the end of the design period yields the balancing volume of $8196 \approx 8200$ m^3.

Conclusions:

- In Alternative A, the reservoirs are located at the suction side of the pumping stations that were designed to operate at similar capacity. Hence, the balancing volume can be shared evenly between the two sources. The total volume per tank, required at the beginning of the design period, is determined as follows:
  - balancing volume: $4400/2 = 2200$ m^3,
  - emergency volume for one (maximum consumption) hour in each tank: $2452.9 \approx 2500$ m^3,
  - other provisions: approximately 0.6 m of the tank height.
    Assuming a circular cross-section of the tank, $D = 35$ m, 0.6 m of water column is equivalent to $577.3 \approx 600$ m^3. The total tank

Table A2.11. Balancing storage volume.

At Year 6			UFW (%) 10			V_{bal} (m^3) 4373.0	
Hour	*pf*	Consumed (m^3/h)	UFW (m^3/h)	Factory (m^3/h)	Demand (m^3/h)	ΔV (m^3/h)	σV (m^3/h)
1	0.68	989.6	110.0		1099.5	571.6	571.6
2	0.65	945.9	105.1		1051.0	620.1	1191.7
3	0.62	902.2	100.2		1002.5	668.6	1860.3
4	0.50	727.6	80.8		808.5	862.6	2722.9
5	0.58	844.0	93.8		937.8	733.3	3456.2
6	0.80	1164.2	129.4		1293.5	377.6	**3833.7**
7	1.08	1571.7	174.6	108.3	1854.6	−183.5	3650.2
8	1.10	1600.8	177.9	108.3	1887.0	−215.9	3434.4
9	1.22	1775.4	197.3	108.3	2081.0	−409.9	3024.5
10	1.35	1964.6	218.3	108.3	2291.2	−620.1	2404.4
11	1.42	2066.4	229.6	108.3	2404.4	−733.3	1671.1
12	1.45	2110.1	234.5	108.3	2452.9	−781.8	889.3
13	1.10	1600.8	177.9	108.3	1887.0	−215.9	673.5
14	0.90	1309.7	145.5	108.3	1563.6	107.5	781.0
15	0.75	1091.4	121.3	108.3	1321.0	350.1	1131.0
16	0.65	945.9	105.1	108.3	1159.3	511.8	1642.8
17	0.73	1062.3	118.0	108.3	1288.7	382.4	2025.2
18	1.00	1455.2	161.7	108.3	1725.3	−54.2	1971.0
19	1.20	1746.3	194.0		1940.3	−269.2	1701.8
20	1.47	2139.2	237.7		2376.9	−705.8	996.0
21	1.45	2110.1	234.5		2344.6	−673.5	322.6
22	1.40	2037.3	226.4		2263.7	−592.6	−270.0
23	1.20	1746.3	194.0		1940.3	−269.2	**−539.2**
24	0.70	1018.7	113.2		1131.9	539.2	0.0
Average	1.00	1455.2	161.7		1671.1		

volume equals 2200 + 2500 + 600 = 5300 m^3. For example, for a height of 5.5 m, $V_{tot} \approx 5290$ m^3. Two equal tanks yield the total storage volume in the system of approximately 10,600 m^3.

- In the case of Alternative B, the balancing volume is located within the network. If the source pumping station works at constant average capacity, the volume of the clear water reservoir in Source 1 will consist mainly of the emergency provision. Thus, at the beginning of the design period:
  - at Source 1, $V_{tot} = 2500 + 600 = 3100$ m^3 (for $D = 30$ m and $H = 4.3$ m, $V_{tot} = 3040$ m^3),
  - for the tank in the system, $V_{tot} = 4400 + 2500 + 600 = 7500$ m^3 ($D = 40$ m, $H = 6.0$ m, $V_{tot} = 7540$ m^3). The total storage volume in the system remains the same as in the case of Alternative A.
- During the design period, the emergency volume will become gradually converted into the balancing volume. If no action is taken, the deficit of the balancing volume 8200 − 4400 = 3800 m^3 would

reduce the initial emergency volume of 5000 m^3 to only 1200 m^3 at the end of the design period. Thus, it is advisable to start extending the volume in good time; otherwise the emergency reserve will become effectively exhausted.

- Based on the demand growth, it is estimated that some additional 10,000 m^3 of the storage volume would be required at the end of the design period. An even share of this volume is planned between the two sources in Alternative A. A preliminary scenario for Alternative B is:
  - in Source 1, the tank volume increases from ± 3100 to ± 4600 m^3,
  - in Source 2, a new clear water tank of ± 4600 m^3 is to be built,
  - the balancing tank volume increases from ± 7500 to ± 11,400 m^3.

Such a large volume of the elevated balancing tank changes the earlier conclusion about its better hydraulic performance compared to the ground tank; to construct a volume of ± 10,000 m^3 at a height of ± 20 m might be a relatively expensive solution, not to mention the aesthetic aspects if the structure is to be located in an urban area.

Preliminary dimensions of all tanks are summarised in Tables A2.12 and A2.13. The dimensions at the end of the design period indicate the total required volumes. Depending on the selected form of the tanks (not necessarily circular cross-section) these dimensions should match the already existing design as much as possible in order to make the volume extension easier (i.e. by adding of the second compartment).

Comments:

- The balancing volume is exclusively dependent from the demand and its pattern of variation. It is a property of a certain distribution area and not of the chosen supply scheme.

Table A2.12. Alternative A – storage tanks.

	Beginning of the design period			End of the design period		
	D (m)	H (m)	V_{tot} (m^3)	D (m)	H (m)	V_{tot} (m^3)
Source 1	35	5.5	5290	50	5.5	10,800
Source 2	35	5.5	5290	50	5.5	10,800

Table A2.13. Alternative B – storage tanks.

	Beginning of the design period			End of the design period		
	D (m)	H (m)	V_{tot} (m^3)	D (m)	H (m)	V_{tot} (m^3)
Source 1	30	4.3	3040	30	6.5	4600
Source 2	—	—	—	30	6.5	4600
Tank	40	6.0	7540	50	6.0	11,780

- In the direct pumping supply schemes, the balancing volume is located at the source(s) i.e. at the suction side of the source pumping stations. There, it evens out the constant production and variable pumping (Alternative A).
- In the combined supply schemes, the balancing volume is located within the distribution area and evens out the constant pumping and variable demand (Alternative B).

Reading:
- Chapter 4, Section 4.2.3: 'Storage design'.

A2.3.6 *Summary of the hydraulic design*

Alternative A – Direct pumping
This alternative comprises:
- network layouts as shown in Figures A2.19 and A2.20 (EPANET input files S6A8 & S30A2),
- selection of the pump units as displayed in Table A2.10,
- storage volume in the system as displayed in Table A2.12.

The following development strategy has been adopted:
- The network is left with larger diameters for reasons of reliability. Part of the reserve capacity will be exhausted before the system is extended.
- Extension of the pumping stations is tentatively planned around Year 15. Laying of new pipes in the system will take place at the same time.
- For practical reasons, the clear water tanks will be immediately constructed with the volumes required at the end of the design period, which also adds to the reliability of supply.

Alternative B – Pumping and balancing storage
The main characteristics of the system in this alternative are:
- network layout as shown in Figures A2.23 and A2.24 (EPANET input files S6B1 and S30B1),
- selection of the pump units as displayed in Table A2.10,
- storage volume in the system as displayed in Table A2.13.

The following development strategy has been adopted:
- The start of the supply from Source 2 is planned in Year 21 at the latest, after the first source reaches its maximum capacity. The additional pipes should be in place at the same time.
- As an additional safety precaution, it is proposed to construct the clear water tank at Source 1 at the beginning with the volume that will be required at the end of the design period.

- The option with the ground level balancing tank has been abandoned due to the pressure problems that cannot be solved by installing a booster station. It is not yet clear either the option with the elevated balancing tank should be accepted due to its excessively large volume and consequently expensive construction.

It appears at this stage that the present topography does not offer an effective way of using the balancing tank in the system, unless its (large) volume is elevated. Initially, this makes Alternative A appear simpler and more straightforward from the hydraulic point of view. Nevertheless, this conclusion is yet to be verified by further analyses of the system operation and a cost calculation.

The third possibility could be to keep the balancing volume at the sources, combined by the water tower with its volume of 500–1000 m^3. The main task of the water tower would be to stabilise the operation of the pumps that in this case do not operate at a constant capacity any more. Such alternative is in essence a combination of A and B.

A2.4 SYSTEM OPERATION

A2.4.1 *Regular operation*

Alternative A – Direct pumping

File S30A3.NET has been created based on the summary in Section A2.3.6. The test simulations of this layout show that sufficient pressure in the network can also be maintained with the pump duty head of 50 instead of 60 mwc at Source 1. With this adjustment, the pump set-up in two sources looks as follows:

- Source 1: 6 units + 1 stand-by, Q_d = 100 l/s, H_d = 50 mwc,
- Source 1: 6 units + 1 stand-by, Q_d = 90 l/s, H_d = 40 mwc.

The simulation starts with three units, 1001, 1002 and 1003, in operation at the first source and two units, 2001 and 2002, at the second source; for all other units, the **Initial Status** in the pump property editor is set as **CLOSED**. By checking the pressures in the system, hour-by-hour, each pump is further switched on or off when the pressure in any node drops outside the range of 20–60 mwc; this operation can be done manually or automatically. It is necessary to keep the pressure as close to 20 mwc for as long as possible.

Both the manual and automatic mode of pump operation are modelled in EPANET in the browser option **Data** >> **Controls** (double-click on **Simple**). For the above-selected pump units, the optimal schedule of manual operation is as follows:

```
---------------------------------------------------------------------
Link      ID       Setting      Condition
---------------------------------------------------------------------
LINK      1003      CLOSED      AT TIME 2
LINK      1003        OPEN      AT TIME 6
LINK      2003        OPEN      AT TIME 6
LINK      1004        OPEN      AT TIME 7
LINK      2004        OPEN      AT TIME 7
LINK      1005        OPEN      AT TIME 9
LINK      2005        OPEN      AT TIME 9
LINK      1006        OPEN      AT TIME 11
LINK      2006        OPEN      AT TIME 11
LINK      1006      CLOSED      AT TIME 13
LINK      2006      CLOSED      AT TIME 13
LINK      1005      CLOSED      AT TIME 13
LINK      2005      CLOSED      AT TIME 13
LINK      1004      CLOSED      AT TIME 14
LINK      2004      CLOSED      AT TIME 14
LINK      1003      CLOSED      AT TIME 15
LINK      1003        OPEN      AT TIME 18
LINK      1004        OPEN      AT TIME 18
LINK      2004        OPEN      AT TIME 18
LINK      1005        OPEN      AT TIME 19
LINK      2005        OPEN      AT TIME 20
LINK      1006        OPEN      AT TIME 20
LINK      2006        OPEN      AT TIME 20
LINK      1006      CLOSED      AT TIME 23
LINK      2006      CLOSED      AT TIME 23
LINK      1005      CLOSED      AT TIME 24
LINK      2005      CLOSED      AT TIME 24
LINK      1004      CLOSED      AT TIME 24
LINK      2004      CLOSED      AT TIME 24
LINK      2003      CLOSED      AT TIME 24
---------------------------------------------------------------------
```

Summarised per pump unit, the same schedule is shown in Table A2.14 (1 is On, 0 is Off). Pumps 1007 and 2007 have been left out of the operation, serving as stand-by units.

Table A2.14. Alternative A: pumping schedule for manual operation (S30A3).

SOURCE 1	Schedule from 0–24	SOURCE 2	Schedule from 0–24
1001	111111111111111111111111	2001	111111111111111111111111
1002	111111111111111111111111	2002	111111111111111111111111
1003	110000111111111000111111 1	2003	000000111111111111111110
1004	000000011111110000111111 0	2004	000000011111110000111111 0
1005	000000000111100000011111 0	2005	000000000111100000001111 0
1006	000000000001100000001110 0	2006	000000000001100000001110 0
1007[1]	000000000000000000000000	2007[1]	000000000000000000000000

[1] Stand-by units.

For such an operation, the ranges of pressures and velocities in the system are shown in Figures A2.36 and A2.37. The supply ratio between the sources is given in Figure A2.38.

Clarification:

- Nodes 11 and 23 indicate the range of pressures in the system; the pressures in all other nodes lay within this range.
- Pipes 08–09 and 03–04 show extreme velocities in the system; the velocities in all other pipes lay within this range.

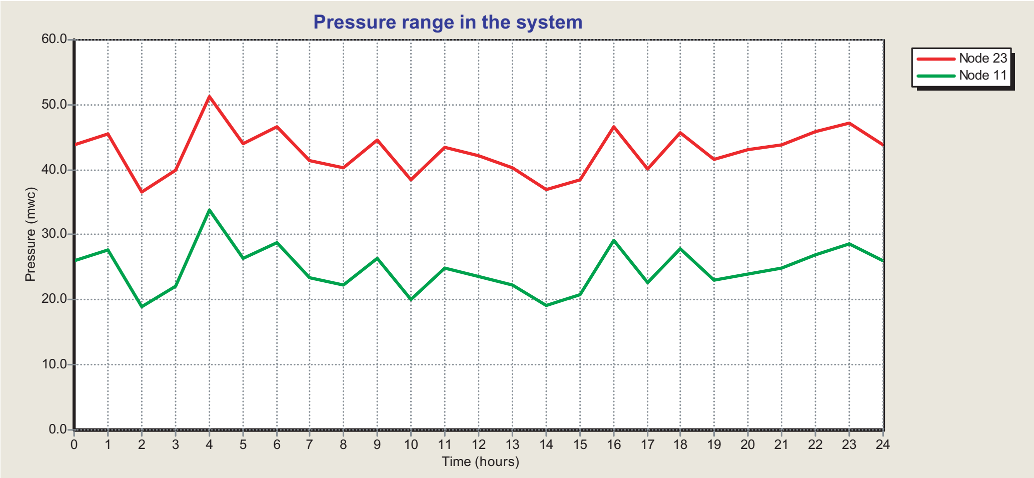


Figure A2.36. S30A3: manual pump operation (schedule in Table A2.14).

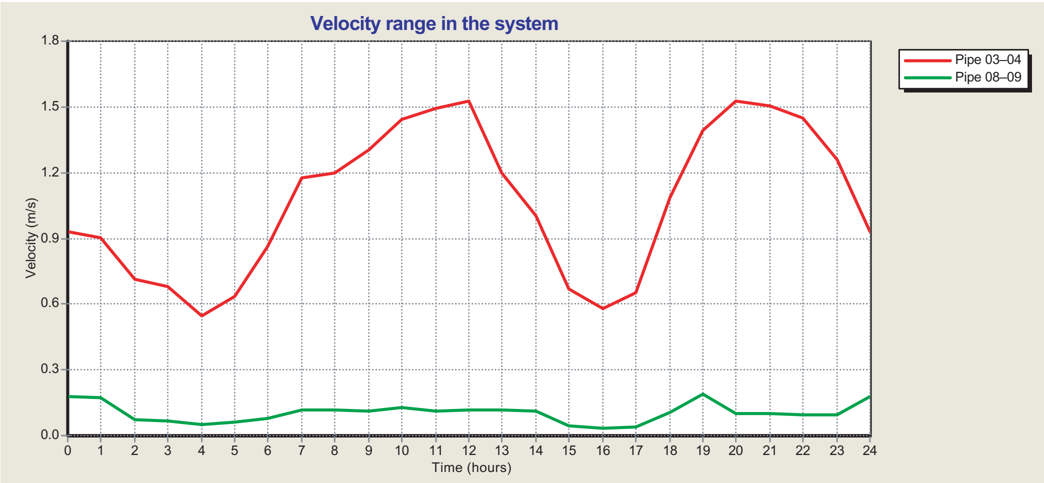


Figure A2.37. S30A3: manual pump operation (schedule in Table A2.14).

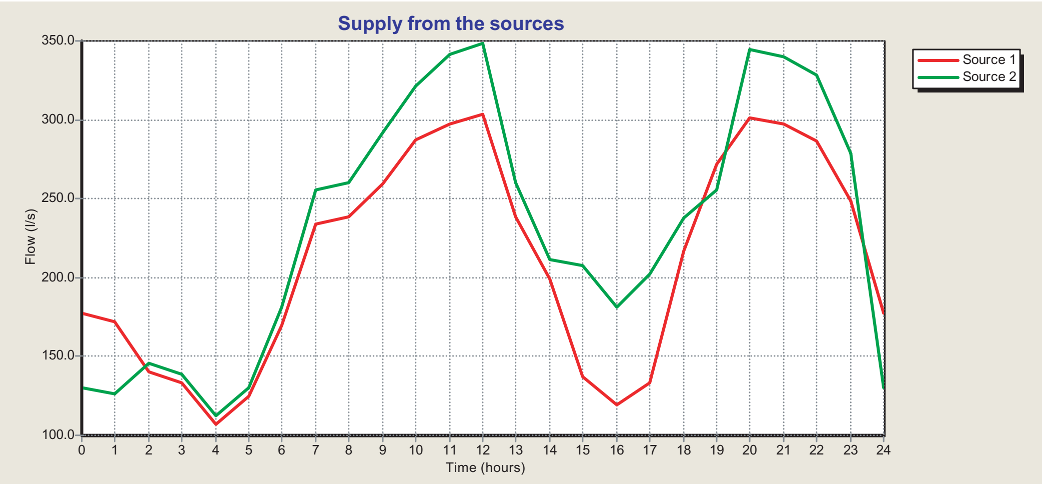


Figure A2.38. S30A3: manual pump operation (schedule in Table A2.14).

Conclusions:

- A fairly balanced supply with stable pressures is provided by the proposed pumping regime.
- In very few cases, pressure slightly below 20 mwc has been tolerated. The minimum pressure of 18.89 mwc occurs in Node 11 at 02:00. This is an hour of low demand and such pressure does not really affect the consumers. Switching on an extra unit at 02:00 in Source 1 would unnecessarily boost the pressure in Node 11 up to 30.01 mwc. Alternatively, an extra pump in Source 2 would give an even higher pressure of 31.59 mwc in Node 11.
- Constantly low velocities in a number of pipes suggest that these should be reduced in diameter unless additional capacity is required for the network operation in irregular situations.

Comments:

- Scheduling of pumps results in a minimum pressure that does not necessarily occur during the maximum consumption hour.
- The minimum pressure criterion should be maintained throughout the entire day. Much higher pressures during the low demand hours are not justified; they cause increased waste of pumping energy and water.
- Choosing the same model of pump for all the installed units has the advantage that the various units can implement the same pumping schedule on different days, which loads them more evenly. The stand-by unit can then also be employed. One possible adaptation of the regime is shown in Table A2.15.

Table A2.15. Alternative A: pumping schedule for adapted manual operation (S30A3).

SOURCE 1	Schedule from 0–24	SOURCE 2	Schedule from 0–24
1001	0000000111111111111111111	2001	0000000111111111111111111
1002	1111111000000111111000000	2002	1111111000000111111000000
1003	1100001111111110001111111	2003	0000001111111111111111110
1004	0000000111111100001111110	2004	0000000111111100001111110
1005	0000000001111000000111110	2005	0000000001111000000111110
1006	1111111000011000000011100	2006	1111111000011000000011100
1007	0000000111111000000111111	2007	0000000111111000000111111

In the automatic mode of operation, the pumps are controlled on the pressure or the water level somewhere in the system. The usual monitoring point is at the discharge of the pumping station. In this exercise, Node 11, as the most critical pressure-wise, has been chosen. After a number of trials, the suggested pump control looks as follows (file S30A4.NET):

```
--------------------------------------------------------------------
Link          ID            Setting          Condition
--------------------------------------------------------------------
LINK          1003             OPEN          IF NODE 11 BELOW 37
LINK          1003           CLOSED          IF NODE 11 ABOVE 47
LINK          1004             OPEN          IF NODE 11 BELOW 32
LINK          1004           CLOSED          IF NODE 11 ABOVE 42
LINK          1005             OPEN          IF NODE 11 BELOW 27
LINK          1005           CLOSED          IF NODE 11 ABOVE 37
LINK          1006             OPEN          IF NODE 11 BELOW 22
LINK          1006           CLOSED          IF NODE 11 ABOVE 32

LINK          2003             OPEN          IF NODE 11 BELOW 37
LINK          2003           CLOSED          IF NODE 11 ABOVE 47
LINK          2004             OPEN          IF NODE 11 BELOW 32
LINK          2004           CLOSED          IF NODE 11 ABOVE 42
LINK          2005             OPEN          IF NODE 11 BELOW 27
LINK          2005           CLOSED          IF NODE 11 ABOVE 37
LINK          2006             OPEN          IF NODE 11 BELOW 22
LINK          2006           CLOSED          IF NODE 11 ABOVE 32
--------------------------------------------------------------------
```

Pumps 1001 and 1002 and 2001 and 2002 are assumed to be constantly in operation. The simulation starts with three pumps switched at each source. This control regime will result in the pump operation as shown in Table A2.16.

For this schedule, the pressure and velocity range in the system are shown in Figures A2.39 and A2.40, and the supply ratio in Figure A2.41.

Clarification:

- The same control node (11) has been selected for both pumping stations for reasons of simplification, which yields the same schedule for both of them.

Table A2.16. Alternative A: pumping schedule for automatic operation (S30A4).

SOURCE 1	Schedule from 0–24	SOURCE 2	Schedule from 0–24
1001	1111111111111111111111111	2001	1111111111111111111111111
1002	1111111111111111111111111	2002	1111111111111111111111111
1003	1111111111111111111111111	2003	1111111111111111111111111
1004	0000001111111111011111110	2004	0000001111111111011111110
1005	0000000111111100000111110	2005	0000000111111100000111110
1006	0000000000111000000011100	2006	0000000000111000000011100
1007	0000000000000000000000000	2007	0000000000000000000000000

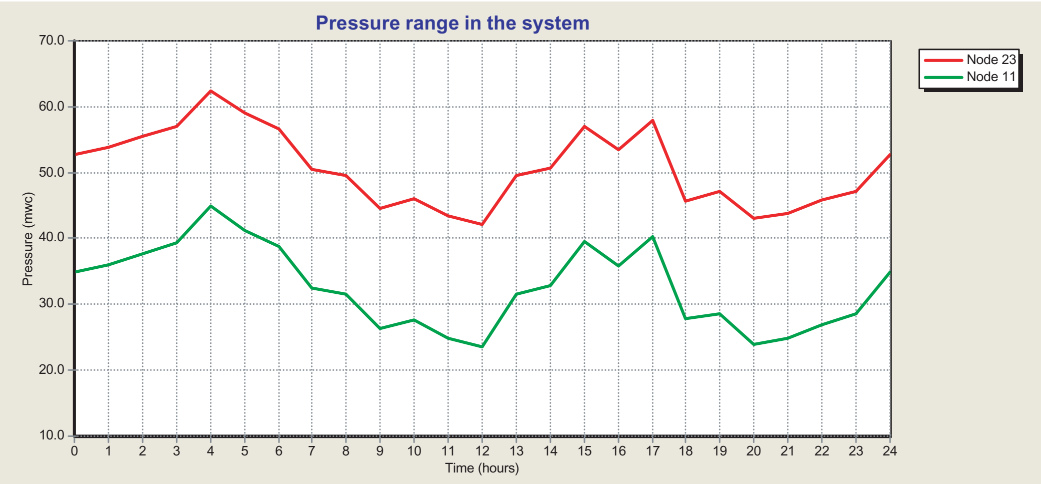


Figure A2.39. S30A4: automatic pump control (schedule in Table A2.16).

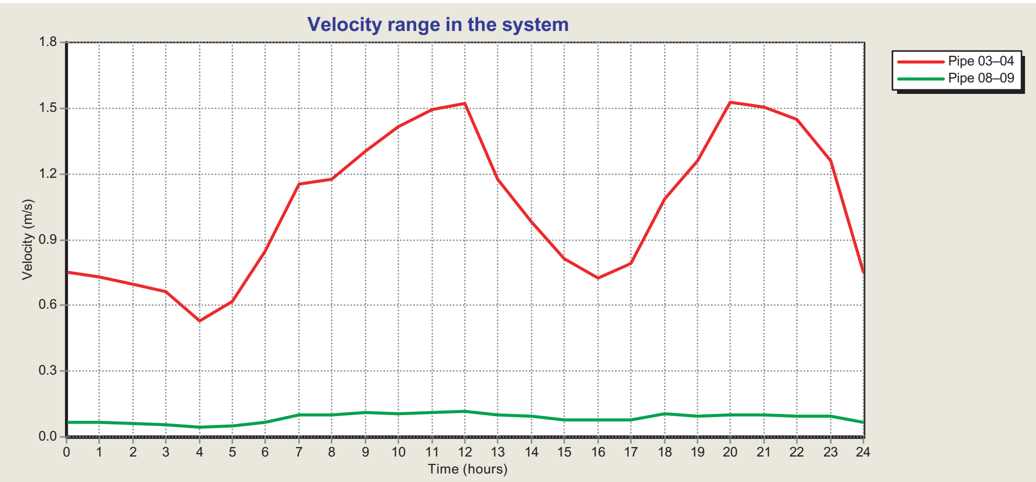


Figure A2.40. S30A4: automatic pump control (schedule in Table A2.16).

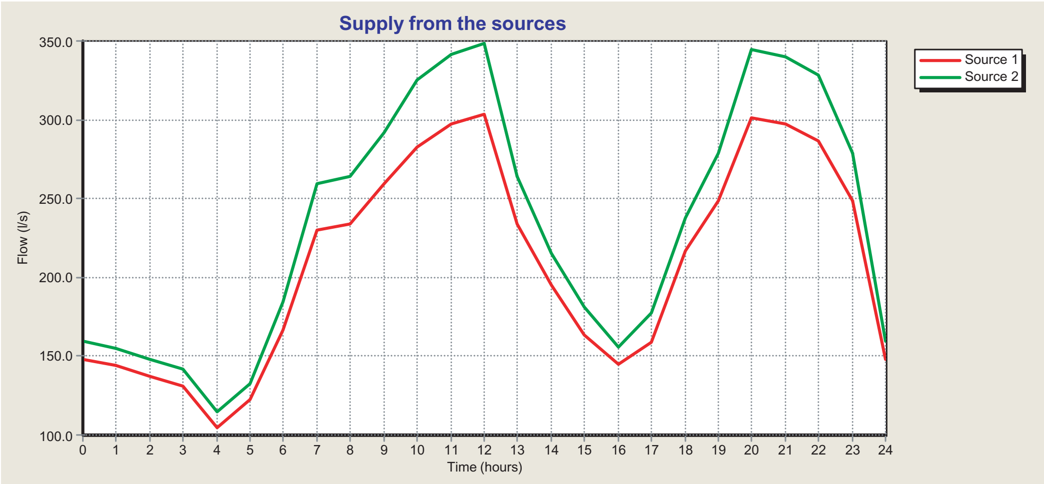


Figure A2.41. S30A4: automatic pumps control (schedule in Table A2.16).

Conclusions:

- For specified controls, the model reacts by maintaining units 1001–1003 and 2001–2003 permanently switched on, while the other pumps are used during the peak demand hours only. To load all pumps evenly, groups 1004–1006 and 2004–2006 could take over the 24-hour operation every second day.
- A larger pressure variation is registered in the nodes over 24 hours than in the case of the manual operation.
- No significant change of the velocities is registered compared to the manual mode of operation.
- The supply ratio is nearly 50–50, which is preferable.

Comments:

- Modelling of the automatic pump operation yields the schedule as a result (as in Table A2.16), while the modelling of the manual pump operation requires such a schedule as an input (Tables A2.14 and A2.15).

The selection of proper control pressures/levels may pose a problem. The following example shows how the model becomes sensitive to even slight modification of the control pressures:

```
-----------------------------------------------------------------------
Link           ID              Setting             Condition
-----------------------------------------------------------------------
LINK           1003               OPEN             IF NODE 11 BELOW 37
LINK           1003             CLOSED             IF NODE 11 ABOVE 42
LINK           1004               OPEN             IF NODE 11 BELOW 32
```

```
LINK        1004     CLOSED     IF NODE 11 ABOVE 37
LINK        1005       OPEN     IF NODE 11 BELOW 27
LINK        1005     CLOSED     IF NODE 11 ABOVE 32
LINK        1006       OPEN     IF NODE 11 BELOW 22
LINK        1006     CLOSED     IF NODE 11 ABOVE 27
LINK        2003       OPEN     IF NODE 11 BELOW 37
LINK        2003     CLOSED     IF NODE 11 ABOVE 42
LINK        2004       OPEN     IF NODE 11 BELOW 32
LINK        2004     CLOSED     IF NODE 11 ABOVE 37
LINK        2005       OPEN     IF NODE 11 BELOW 27
LINK        2005     CLOSED     IF NODE 11 ABOVE 32
LINK        2006       OPEN     IF NODE 11 BELOW 22
LINK        2006     CLOSED     IF NODE 11 ABOVE 27
---------------------------------------------------
```

Figure A2.42 shows the pressures in the system (S30A5.NET).

Clarification:

- The diagram shows different pressure at 0 and 24 hours although the demand level is the same. The reason is the operation at 0 hours that is defined through the initial status of the pumps, while the operation at 24 hours depends on the specified control of pumps.

Conclusions:

- Unlike in S30A4, there is no overlap of the On and Off settings in the [CONTROLS] section of S30A5. This is the likely cause of the unstable operation and the low pressure in the system.

Simulations of the automatic pump operation may also show different results depending on the selected time step of the calculation. With the

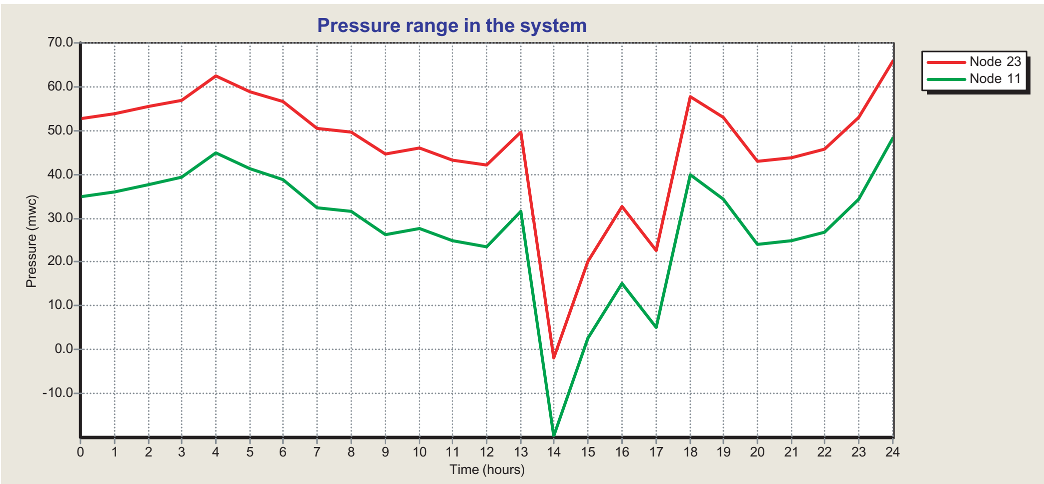


Figure A2.42. S30A5: modified settings of automatic pump operation.

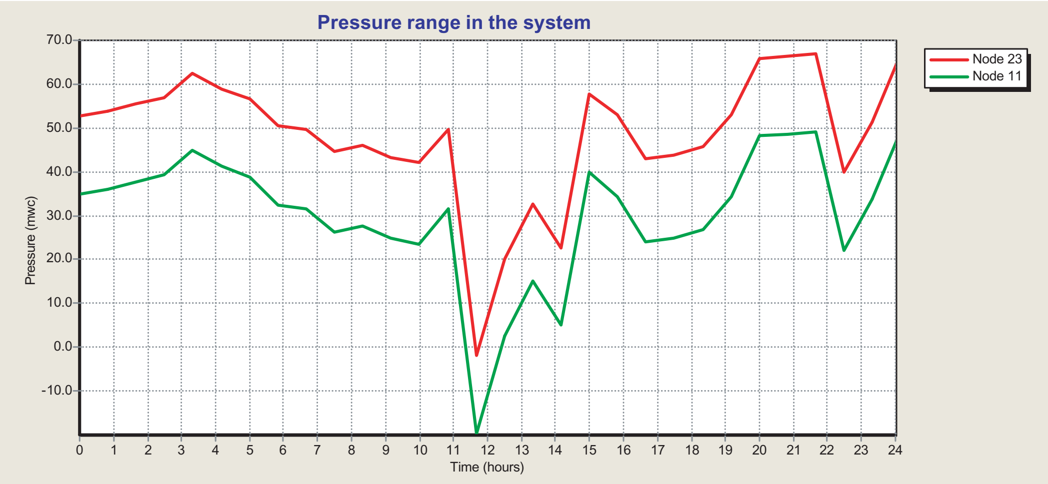


Figure A2.43. S30A6: automatic pump operation with time steps of 1/2 hour.

same controls as in S30A5, the hydraulic time step and the demand pattern step have been reduced from 1 hour to 30 minutes (the browser option **Data** >> **Options** >> **Times** (click on **Hydraulic/Pattern Time Step**). Linear interpolation of the hourly peak factors has been applied resulting in 48 values, each for every 30 minutes. The pressure in the system is indicated in Figure A2.43.

Alternative B – Pumping and balancing storage

The simulation starts with the same system components as listed in Section A2.3.6 (input file S30B2.NET). As in the case of Alternative A, the pump duty head of 50 mwc has been adopted in Source 1. After a number of trial simulations, the tank $D = 50$ m with a depth of 6.5 m, slightly above the original estimate, has been elevated to 22 m (50 msl). Furthermore:

- The pump suction water levels in 101 and 201 have been set at ground level (modelled as reservoirs with a **Total Head** of 10.7 and 25.0 msl, respectively).
- The minimum depth of the elevated tank in 301 is 2.5 m and at the beginning of simulation the depth is estimated to be 3 m.
- The pumping stations are designed with the following units:

```
[PUMPS]
-----------------------------------------------------------------------
ID      Suction    Pressure    Duty          Duty
        Node       Node        Head (mwc)    Flow (l/s)
-----------------------------------------------------------------------
1001    101        102         50            150          ; PST1 - unit 1
1002    101        102         50            150          ;      - unit 2
```

```
1003   101      102      50        150        ;      - unit 3
1004   101      102      50        150        ;      - unit 4
1005   101      102      50        150        ;      - unit 5
1006   101      102      50        150        ;      - unit 6
1007   101      102      50        150        ; PST1 - stand-by
2001   201      202      40        130        ; PST2 - unit 1
2002   201      202      40        130        ;      - unit 2
2003   201      202      40        130        ; PST2 - stand-by
----------------------------------------------------------------
```

All pumps in both sources, except the stand-by pumps, are switched on and operate for 24 hours at a more or less constant regime i.e. average flow; the demand variation is balanced from the tank. Consequently, no control commands are required for the pumps in this mode of operation. The pressure range in the system is shown in Figure A2.44 and the tank water depth variation in Figure A2.45.

Conclusions:

- The system operation is stable. The minimum pressure of 20.80 mwc appears in Node 11 at 12:00. Switching a few pumps off during the periods of high pressure (low demand) can further reduce the pressure. For example, operating four instead of six pumps in Source 1 until 06:00 will have no significant implication on the patterns in the above two figures except that the pressure will slightly drop (in Node 11 at 12:00, p = 20.47 mwc).

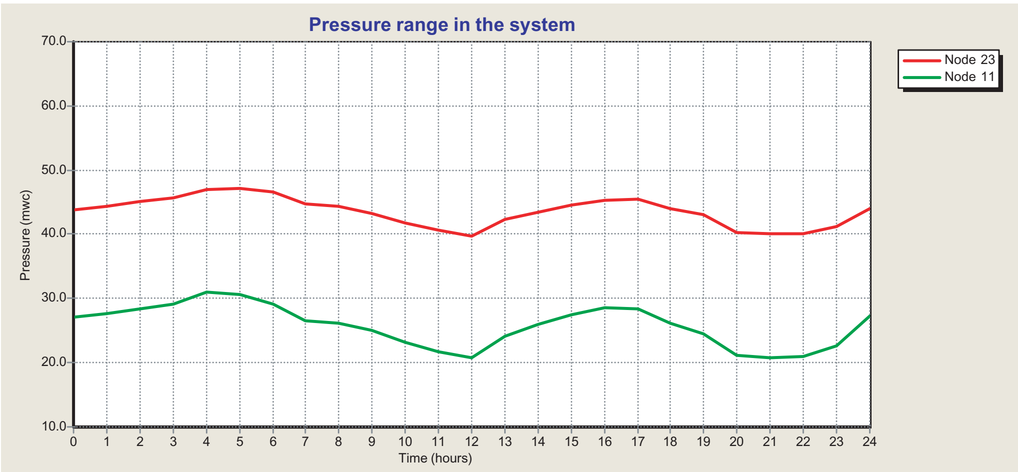


Figure A2.44. S30B2: 22 m-high balancing tank, D = 50 m, H = 6.5 m.

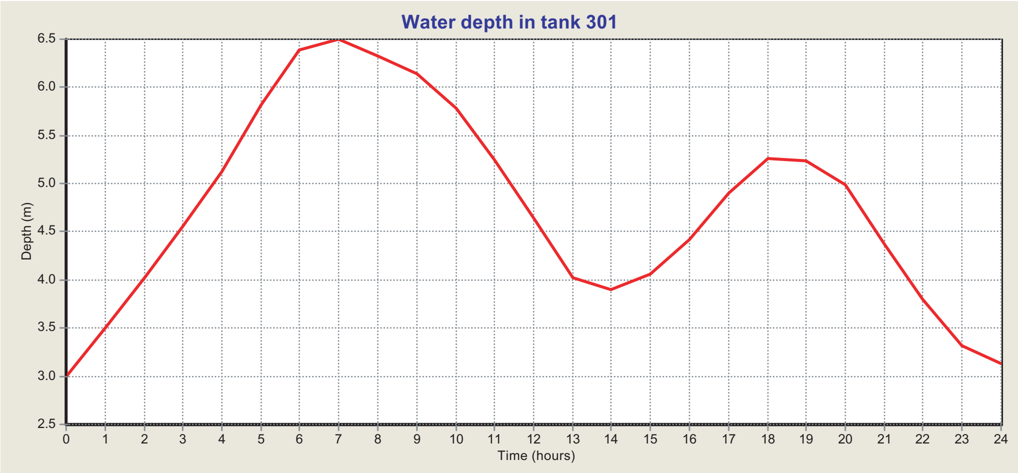


Figure A2.45. S30B2: 22 m-high balancing tank, $D = 50$ m, $H = 6.5$ m.

A further test has been made with smaller pumps and an elevated tank of 20 m (S30B3.NET):

```
[PUMPS]
-----------------------------------------------------------------------
ID       Suction     Pressure     Duty            Duty
         Node        Node         Head  (mwc)     Flow  (l/s)
-----------------------------------------------------------------------
1001          101          102          50          130          ;
150- > 130
1002          101          102          50          130
1003          101          102          50          130
1004          101          102          50          130
1005          101          102          50          130
1006          101          102          50          130
1007          101          102          50          130
2001          201          202          40          120
2002          201          202          40          120
2003          201          202          40          120
-----------------------------------------------------------------------
```

The results are shown in Figures A2.46 and A2.47.

Conclusions:

- The system is stable although the pressure drops slightly below 20 mwc. The most critical value is 18.53 mwc in Node 11 at 12:00.

Comments:

- For the given network layout, good operation of the balancing tanks is reached as a result of matching the pump operation with the

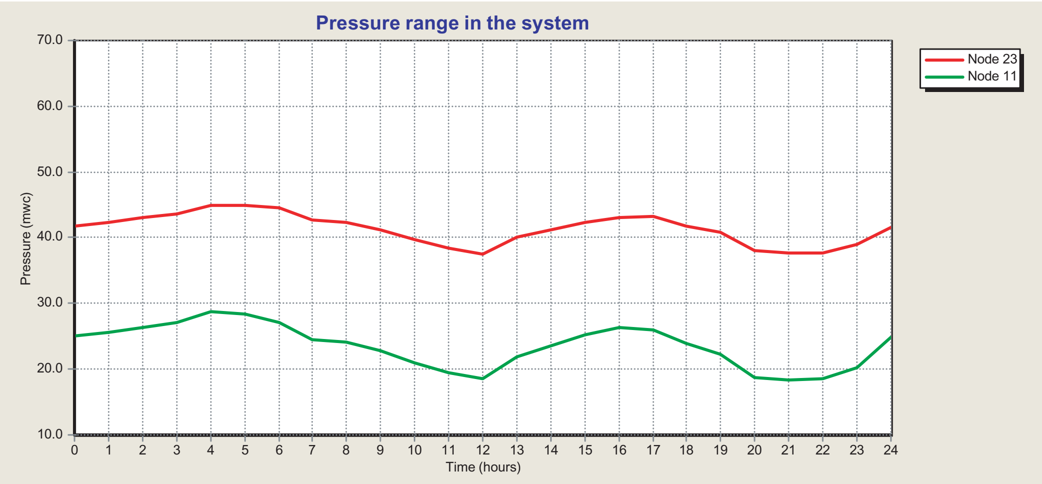


Figure A2.46. S30B3: smaller pumps and 20 m-high tank, $D = 50$ m, $H = 6.5$ m.

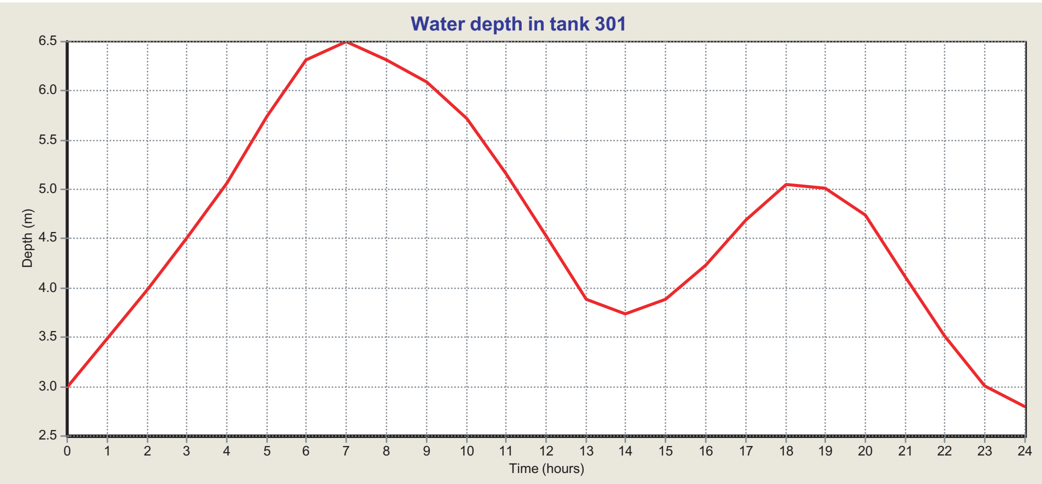


Figure A2.47. S30B3: smaller pumps and 20 m-high tank, $D = 50$ m, $H = 6.5$ m.

tank elevation and its size. While doing this, the following three cases are possible:

CASE 1 – The tank receives too much water: This situation is simulated in the file S30B3-1.NET by putting the balancing volume 10 m lower, i.e. from 20 to 10 m height. The water depth variation of the tank is shown in Figure A2.48. It is clear from the figure that the tank will overflow by the next day already if the same operation is maintained. The

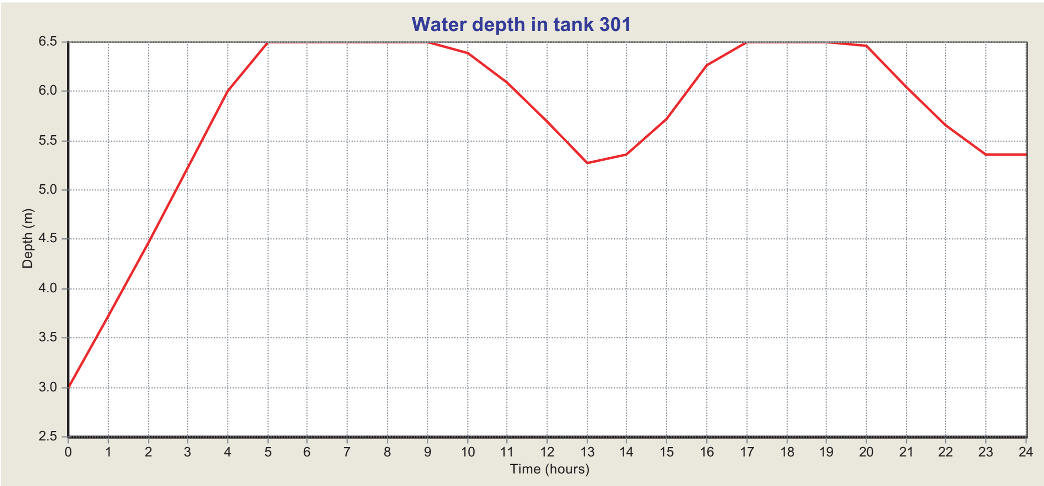


Figure A2.48. S30B3: 10 m-high balancing tank, $D = 50$ m, $H = 6.5$ m (CASE 1).

following design options are suggested provided sufficient pressure can be maintained in the system:

- the tank bottom should be raised to higher elevation, or/and
- smaller pumps should be used at the sources, or/and
- some pumps should be switched off in periods of low demand.

CASE 2 – The tank loses too much water: This is simulated in S30B3-2.NET by setting the balancing volume 30 m high, instead of 20 m. The water depth variation in the tank resulting from this action is shown in Figure A2.49. This case contrasts with the above.

CASE 3 – The tank is balancing too quickly: To demonstrate this, the volume of the tank has been reduced from $D = 50$ m to $D = 25$ m, keeping the elevation at $H = 20$ m. The results are shown in Figure A2.50 (S30B3-3.NET). The remedies are:

- to increase the tank volume i.e. the cross-section area (diameter) or/and the available depth,
- to adjust the pumping schedule; if the balancing volume has been reduced, the pumps cannot operate at constant (average) flow any more.

When the elevated tank is reduced in volume, it loses the demand balancing function and becomes in fact a water tower.

Water towers provide stable pressures in the system and at the same time prevent too frequent switching of the pumps. The pumping schedule in this case becomes similar to those for the direct supply conditions. To satisfy the pumping flow that exceeds Q_{avg} in some periods of the day,

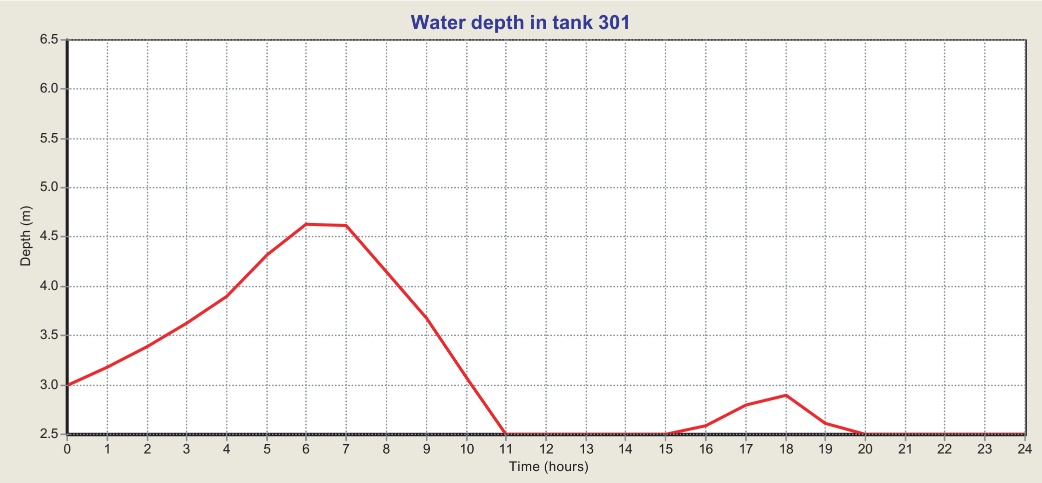


Figure A2.49. S30B3: 30 m-high balancing tank, D = 50 m, H = 6.5 m (CASE 2).

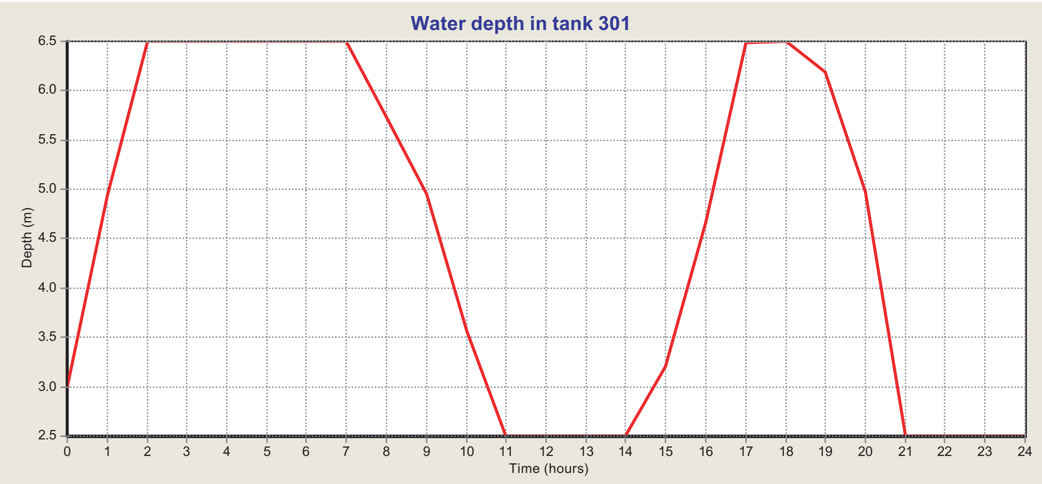


Figure A2.50. S30B3: 20 m-high balancing tank, D = 50 m, H = 6.5 m (CASE 3).

stronger pumps are needed than those operated in combination with balancing tanks.

Usually, the pump operation will be controlled automatically depending on the registered water level in the tank. To simulate these conditions, a 20 m-high water tower of diameter 12 m, minimum and maximum depth of 0.2 and 6.5 m, respectively (making the volume approximately 730 m^3) has been simulated in file S30B4.NET. The initial water depth at the beginning of the simulation was set at 1.0 m and the following pump control is suggested by taking the same pumps from file S30B2.NET:

```
[CONTROLS]
------------------------------------------------------------------------
Link        ID          Setting        Condition
------------------------------------------------------------------------
LINK        1003        OPEN           IF NODE 301 BELOW 4.0
LINK        1003        CLOSED         IF NODE 301 ABOVE 6.0
LINK        1004        OPEN           IF NODE 301 BELOW 3.0
LINK        1004        CLOSED         IF NODE 301 ABOVE 5.0
LINK        1005        OPEN           IF NODE 301 BELOW 2.0
LINK        1005        CLOSED         IF NODE 301 ABOVE 4.0
LINK        1006        OPEN           IF NODE 301 BELOW 1.0
LINK        1006        CLOSED         IF NODE 301 ABOVE 3.0
LINK        2002        OPEN           IF NODE 301 BELOW 3.0
LINK        2002        CLOSED         IF NODE 301 ABOVE 6.0
LINK        2003        OPEN           IF NODE 301 BELOW 2.0
LINK        2003        CLOSED         IF NODE 301 ABOVE 5.0
LINK        2004        OPEN           IF NODE 301 BELOW 1.0
LINK        2004        CLOSED         IF NODE 301 ABOVE 4.0
------------------------------------------------------------------------
```

Two pumps at Source 1 and one pump at Source 2 are switched on at the beginning of the simulation. Based on the above controls, the pumps will operate further according to the schedule shown in Table A2.17. As a consequence of the control regime, two additional units at Source 2 will be needed.

The pressure range in the system is shown in Figure A2.51, while the volume variation in the water tower is shown in Figure A2.52.

Conclusions:

- More sudden changes of the water tower depth are caused by the pump operation. This has no serious implications on the pressure in the system.
- The level in the tank at the end of the day does not match the level at the beginning of the day. This is not a problem because the tank does

Table A2.17. Alternative B: pumping schedule for automatic operation controlled from the water tower (S30B4).

SOURCE 1	Schedule from 0–24	SOURCE 2	Schedule from 0–24
1001	1111111111111111111111111	2001	1111111111111111111111111
1002	1111111111111111111111111	2002	1000000111111111001111111
1003	1011100011111111101111111	2003	1000000011111110000011111
1004	1000000111111110001111110	2004	1000000000011100000001110
1005	1000000010111100000011110	2005	0000000000000000000000000
1006	1000000000011100000001100		
1007	0000000000000000000000000		

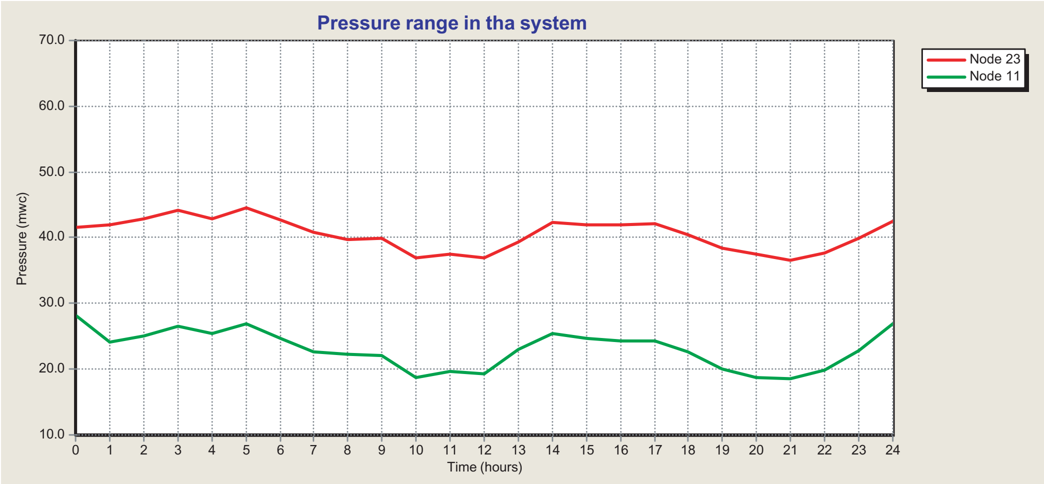


Figure A2.51. S30B4: 20 m-high water tower, $V = 730\ m^3$.

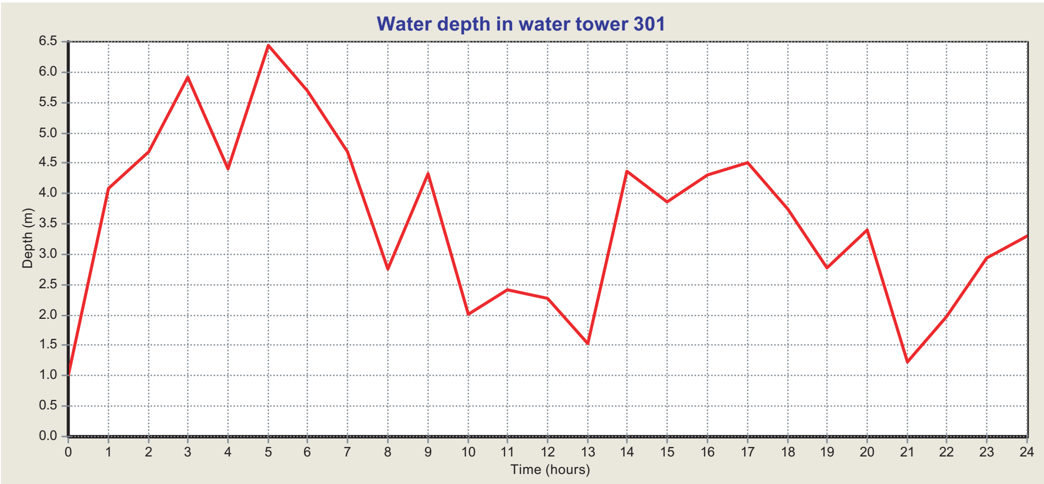


Figure A2.52. S30B4: 20 m-high water tower, $V = 730\ m^3$.

not have a demand-balancing role. In an automatic mode of operation switching the pumps on or off can adjust any extremely low or high level.

A2.4.2 *Factory supply under irregular conditions*

One of the design requirements is to provide a reliable supply for the factory. Under normal supply conditions the minimum pressure at that point is ± 27 mwc, in both alternatives. Obviously, the most critical situation will be caused by the pipe burst 01–16, which is rather easy to solve because another pipe in parallel could be added to it. More difficult to predict are the consequences of the failure of other pipes connecting Node 16. Furthermore, the supply requirement during fire fighting in the factory has been examined in this section.

In all cases it is assumed that the disaster takes place during the maximum consumption hour. As a remedy, both pumping stations are set at full capacity, including the stand-by units.

Alternative A – Direct pumping

The regular operation of the S30A3 layout is shown in Figure A2.53. The failure of pipes 15–16, 16–24 and 16–17 respectively are shown in Figures A2.54–A2.56.

Clarification:

- While simulating a pipe failure, it is assumed that the line is closed i.e. being repaired. This is an acceptable approach bearing in mind that the real quantities of water lost from the system are difficult to predict. Hence, the pipe should be simply disconnected from the rest of the

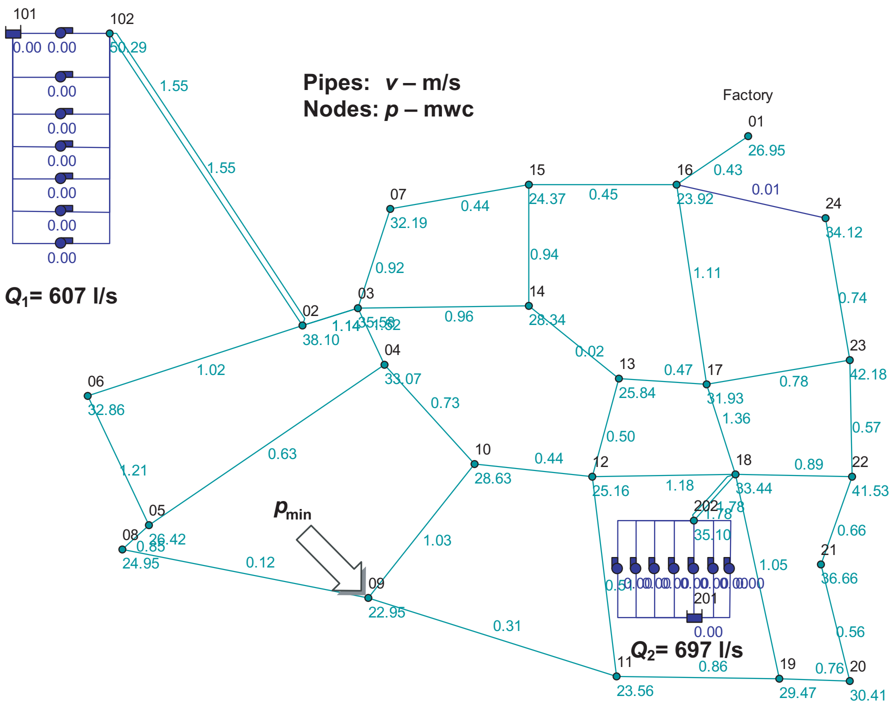


Figure A2.53. S30A3: manual operation at 12:00.

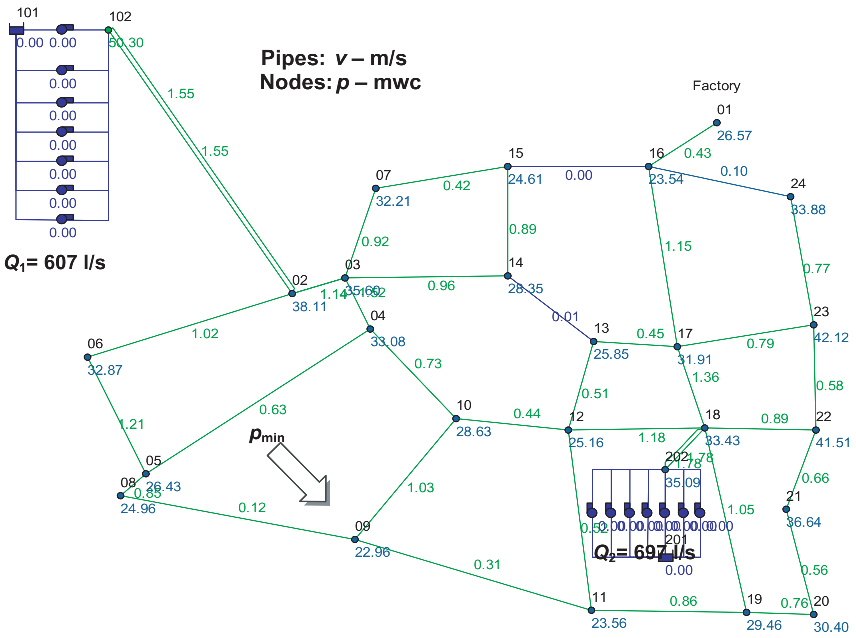


Figure A2.54. S30A3-1: burst 15-16, operation at 12:00.

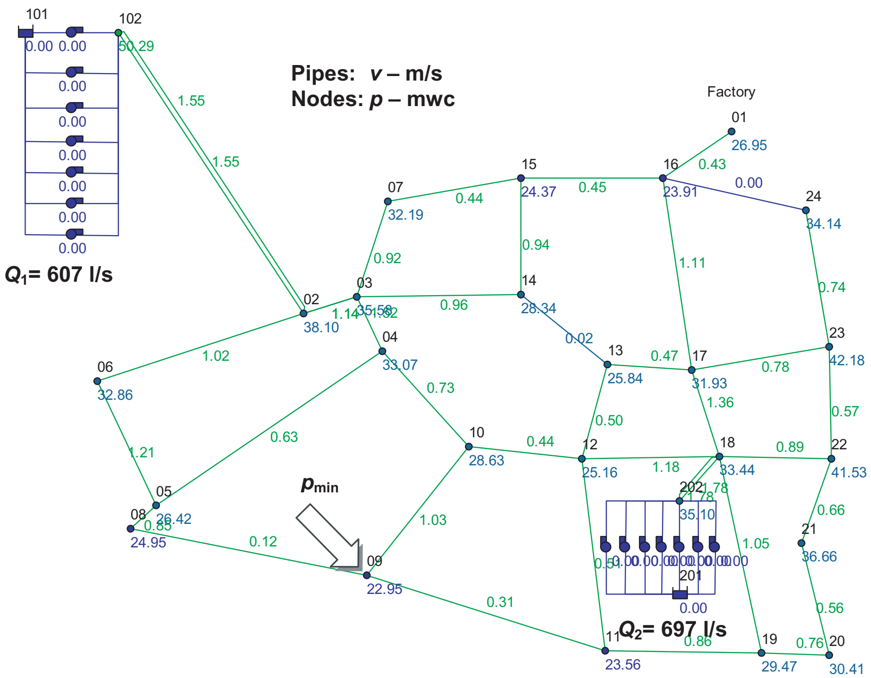


Figure A2.55. S30A3-2: burst 16-24, operation at 12:00.

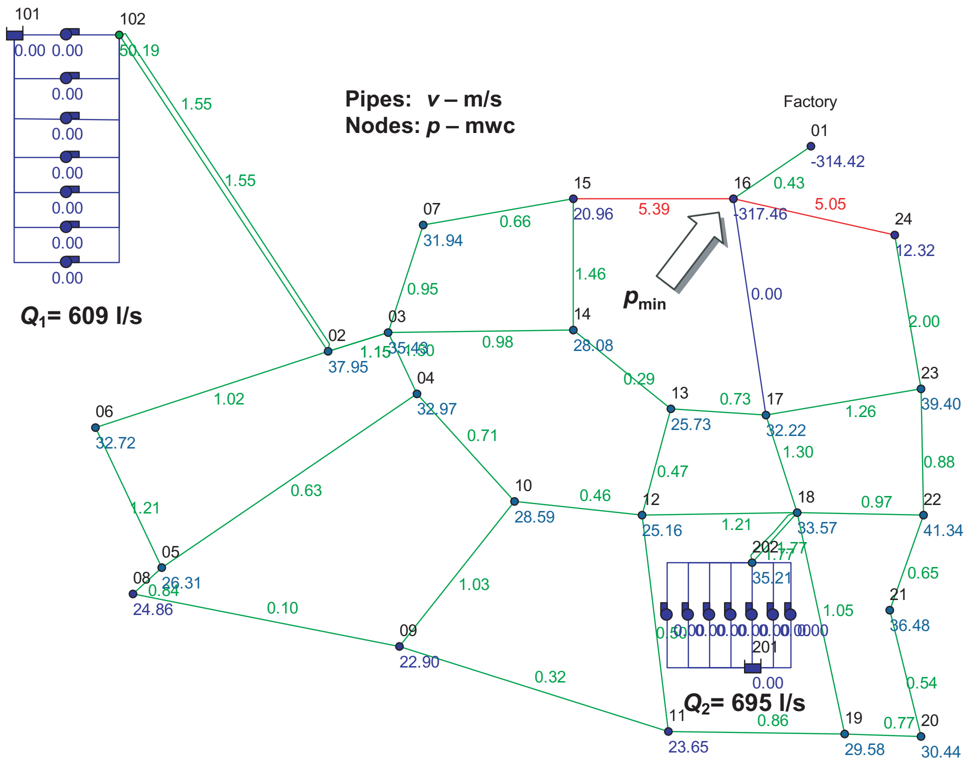


Figure A2.56. S30A3-3: burst 16-17, operation at 12:00.

system by changing the **Initial Status** in its property editor from **OPEN** to **CLOSED**, or writing the corresponding command in the Control Editor.

Conclusions:

- There is hardly any effect on pressure in the system as a result of the burst of pipes 15-16 and 16-24. These are small diameters pipes (D = 100 mm) with low flows during regular operation and can be disconnected without affecting the rest of the system.
- Failure of pipe 16-17 is a much more serious problem. This pipe is on the main path from Source 2 and if closed, the water will move to the alternative routes towards the factory. In this case, pipes 15-16-24 become much too small and a severe pressure drop (velocities above 4 m/s!) will be caused. Increasing the pumping head at the sources does not help and enlarging of at least one of the pipes will be necessary.

Figure A2.57 shows the solution by replacing the diameter of 15-16 with D = 300 mm. In both pumping stations, the stand-by units are switched on.

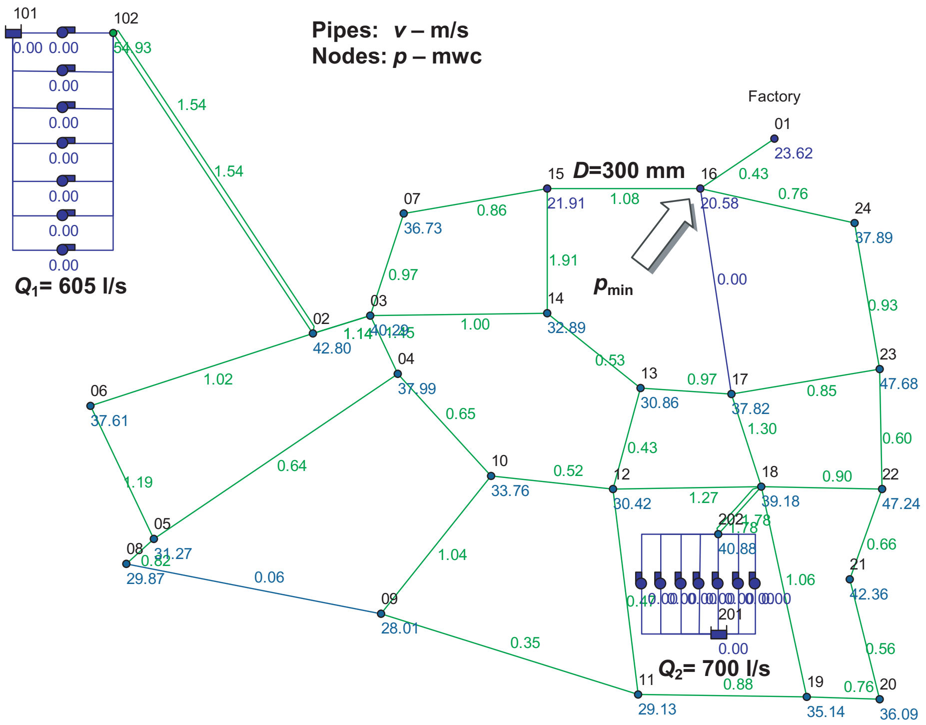


Figure A2.57. S30A3-4: remedy, operation at 12:00.

Conclusions:

- The pressure in critical points has been restored at a somewhat lower level, however it is still above the required minimum of 20 mwc.

To simulate fire demand at the factory node, an additional quantity of 180 m^3/h = 50 l/s has been assigned to Node 01 (total 80.09 l/s) in the S30A3 layout. The results of the calculation are shown in Figure A2.58.

Conclusions:

- As a result of the increased demand in Node 01, the pressure in this node drops to 16 mwc, while a minimum 30 mwc has been stipulated. Except for nodes 01 and 16, the rest of the system is not affected; the bottleneck is again somewhere in the area nearby the factory.

Replacing pipes 07-15-16 with diameter D = 300 mm can solve the problem. An operation with this modification of the system layout is shown in Figure A2.59. The stand-by pumps in both pumping stations are turned On.

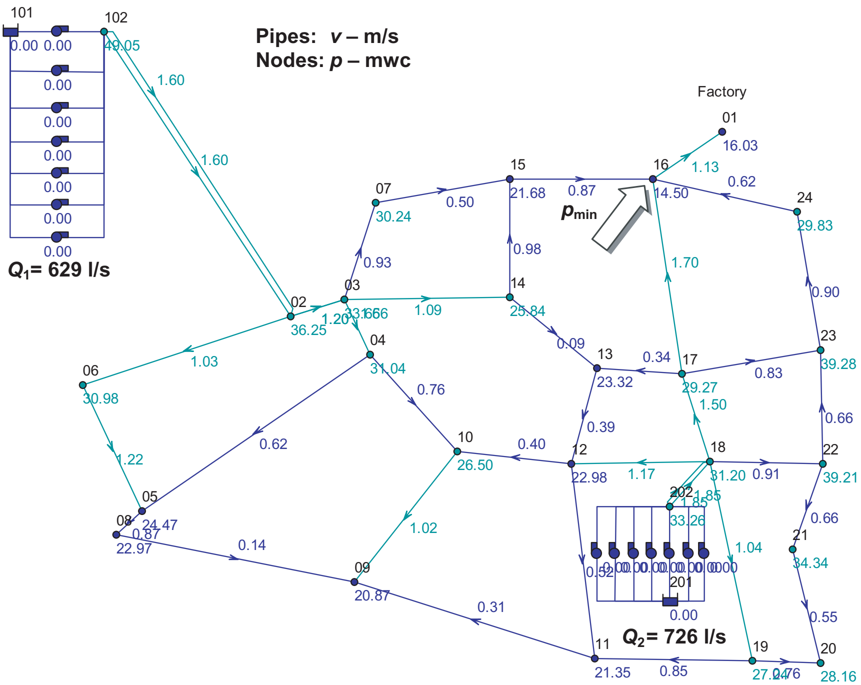


Figure A2.58. S30A3: fire demand at the factory, operation at 12:00.

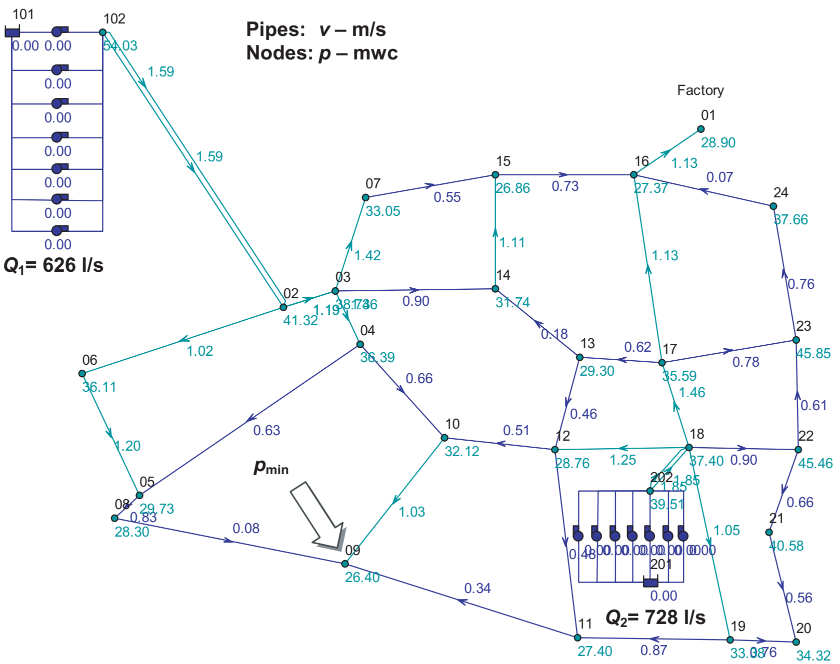


Figure A2.59. S30A3: remedy for fire demand, operation at 12:00.

Conclusions:

- The pressure in the factory is slightly below the required 30 mwc (28.90 mwc). Further computer runs would show that reaching the minimum of 30 mwc requires an additional pumping unit in both pumping stations. This is considered to be rather an expensive option in relation to the gain in the pressure.

Comments:

- It can be assumed that the fire is limited to a certain number of hours. The fire demand specified in this exercise is slightly exaggerated for educational purpose i.e. to realise the impact on the rest of the network. Pressure problems during fire fighting can also be solved by local measures, such as the operation of district valves, booster installation at the factory, etc. It is normal to assume that consumers in the vicinity of the object will be temporarily affected during fire fighting. Nonetheless, the inconvenience caused during a few hours is usually acceptable compared to a large investment in pipes and pumps that would rarely function.

Alternative B – Pumping and balancing storage

The simulation of disasters in distribution systems with balancing tanks takes into consideration the tank level at the moment of disaster. Obviously, the balancing pattern of the tank is going to be disrupted, which may affect the pressure in the system.

The pipe burst near the factory has been simulated keeping the 20 m-high balancing tank and manually operated pumps (file S30B3.NET). The regular operation is shown in Figure A2.60.

The same burst scenarios have been analysed as in the case of Alternative A. To consider the level in the tank, the bursts have been assumed to occur during the maximum consumption hour, with the stand-by pumps switched on at the same moment. The sample input format for the control lines to simulate such operation is as follows:

```
[CONTROLS]
-------------------------------------------------------------------
Link      ID        Setting       Condition
-------------------------------------------------------------------
LINK        31      CLOSED        AT TIME 12
LINK      2003      OPEN          AT TIME 12
-------------------------------------------------------------------
```

Clarification:

- Figures A2.61, A2.63 and A2.65 show the effect of the burst that happened at 12:00, while Figures A2.62, A2.64 and A2.66 show the situation after switching on the stand-by pump at the second source, also at 12:00 hours.

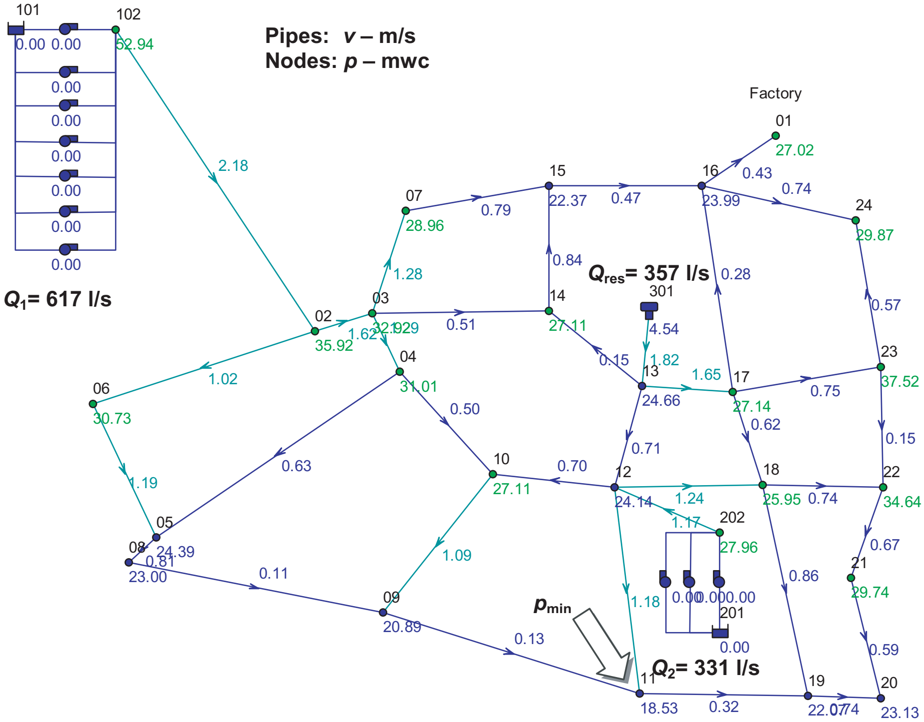


Figure A2.60. S30B3: elevated tank, operation at 12:00.

Conclusions:

- In none of the cases did the burst cause severe pressure problems. Pipes 15-16-17 already belong to the secondary mains loop and further enlargement is therefore not required. Due to sufficient reserve capacity in the system, the stand-by pump at Source 2 is able to easily restore the minor drop of pressure.

Fire demand at the factory is analysed in a similar way as the pipe burst, assuming that the fire breaks out exactly during the maximum consumption hour. To achieve this condition in the model, the demand pattern of the factory has been adjusted to simulate the disaster at 12:00 (the peak factor has been changed from 1.0 to 2.66, which increases the regular factory demand from approximately 30 to 80 l/s). The simulation results are shown in Figure A2.67.

Conclusions:

- No serious loss of pressure will be observed in the system. However, the pressure in the factory itself drops significantly below the required 30–23.98 mwc; thus some remedying is necessary. In the first place

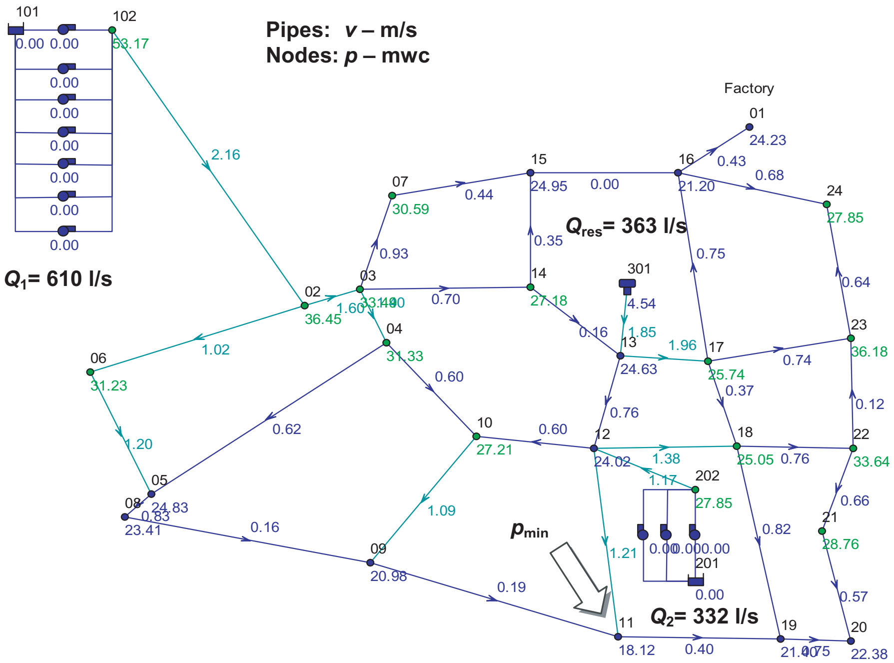


Figure A2.61. S30B3: burst 15-16, operation at 12:00.

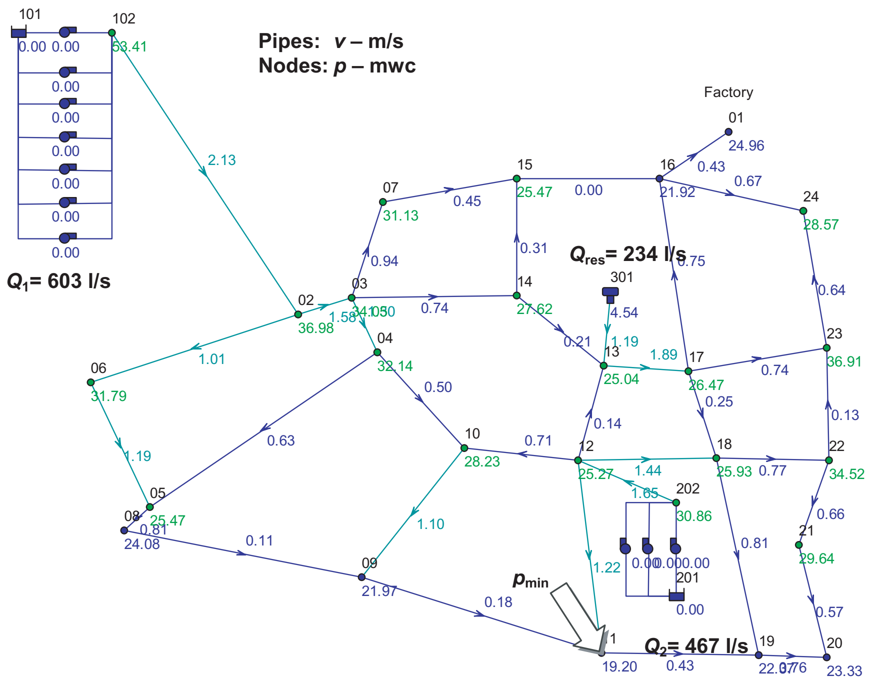


Figure A2.62. S30B3: remedy, operation at 12:00.

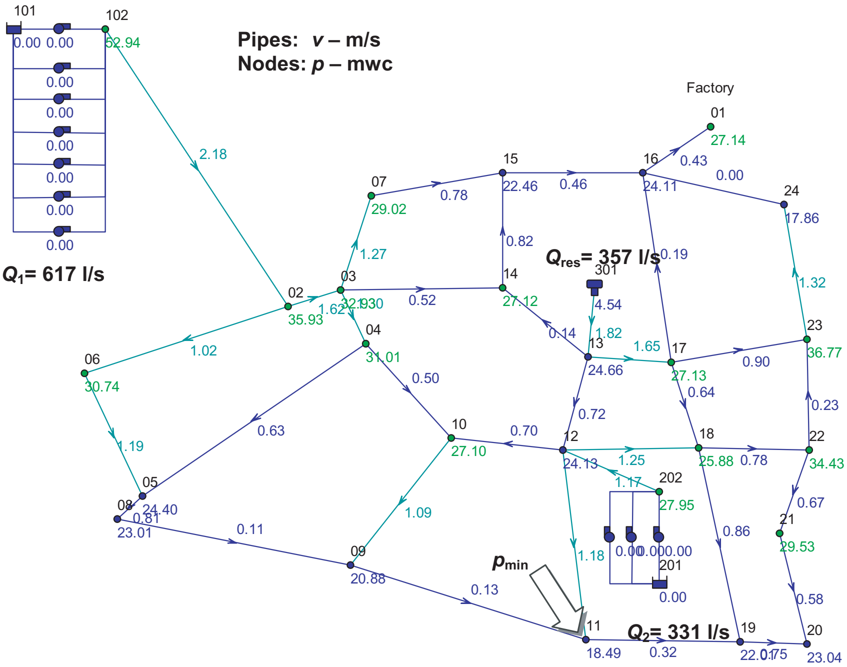


Figure A2.63. S30B3: burst 16-24, operation at 12:00.

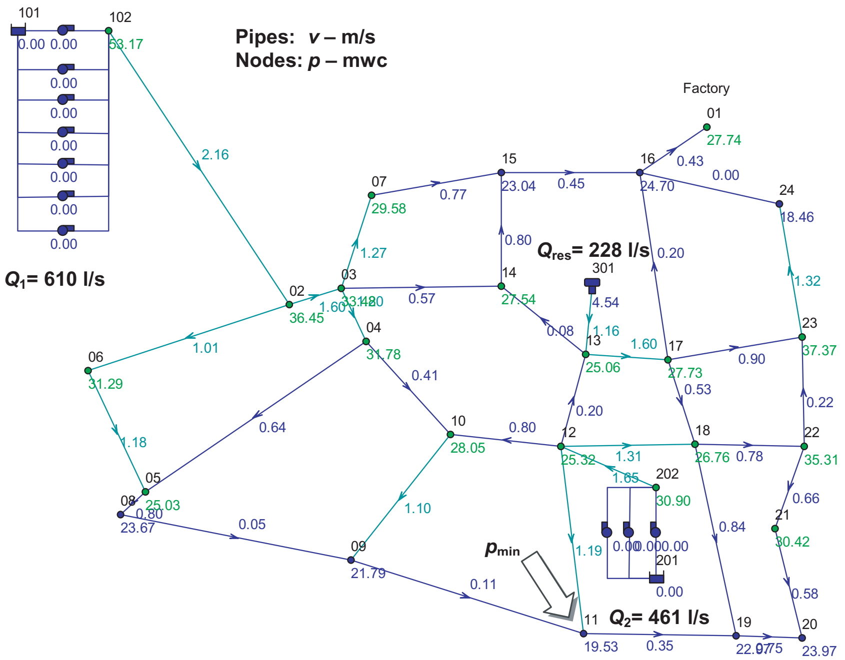


Figure A2.64. S30B3: remedy, operation at 12:00.

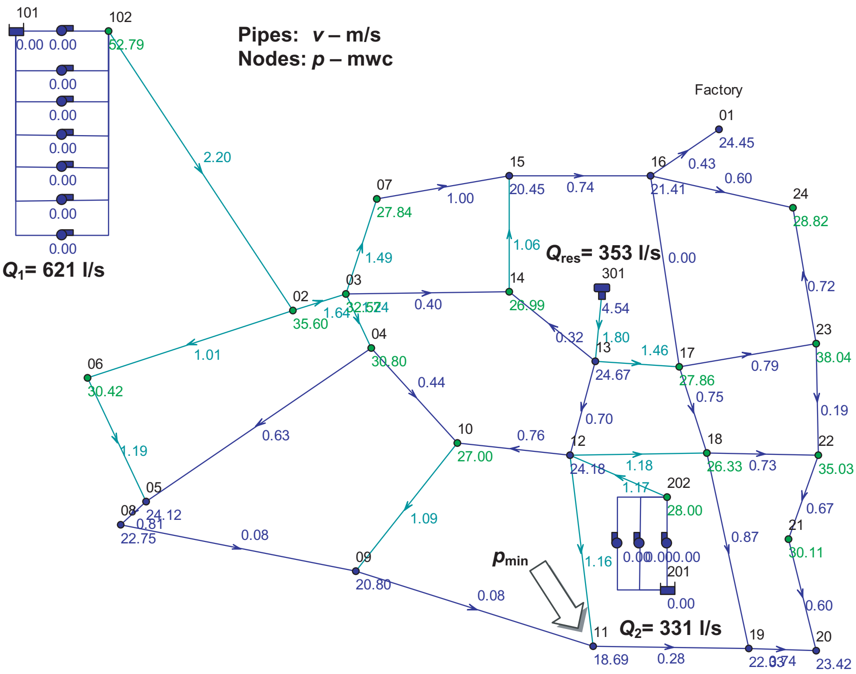


Figure A2.65. S30B3: burst 16-17, operation at 12:00.

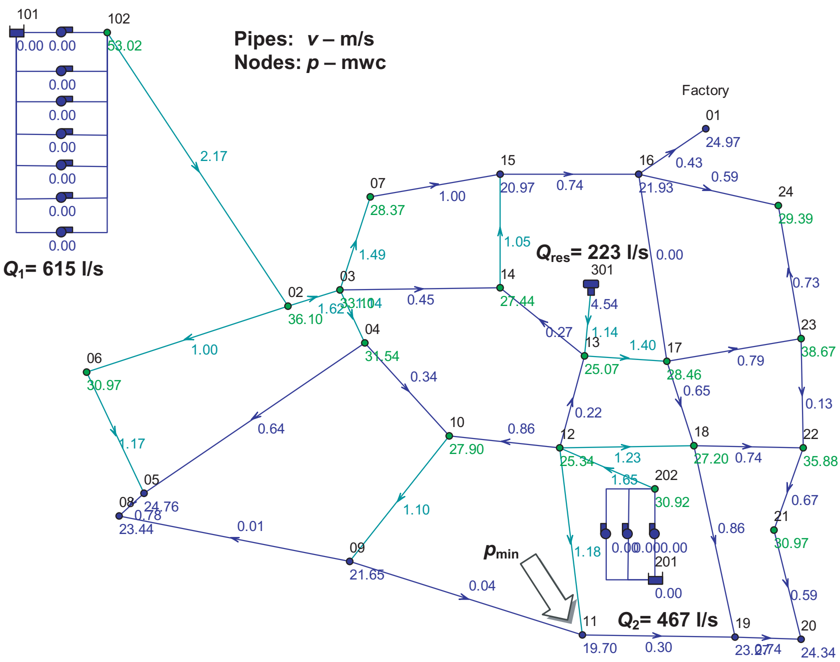


Figure A2.66. S30B3: remedy, operation at 12:00.

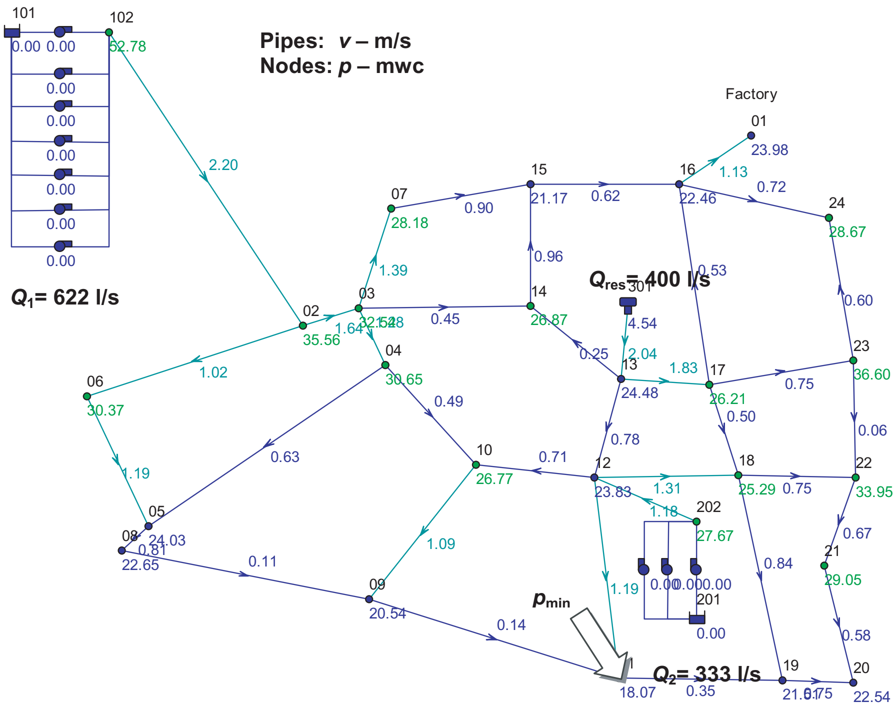


Figure A2.67. S30B3: fire demand, operation at 12:00.

the pipes 01-16 and 13-17 should be enlarged from 300 to 400 and 400 to 500 mm, respectively. In particular, the second pipe was generating a large velocity (head-loss) and this measure was meant to reduce its resistance. In addition, two extra pumps should be added to the pumping station in Source 2 (total 4 + 1). This remedy, shown in Figure A2.68, brings the pressure in the factory close to 30 mwc, but is altogether complex and rather expensive.

A better solution would have been reached if, instead of two extra units in Source 2, a parallel pipe of 600 mm had been laid next to 102-02, and the stand-by pump in Source 1 switched on. The pressure would have grown substantially but the maximum supply capacity of Source 1 would have been exceeded in this situation. This solution, shown in Figure A2.69, is therefore unacceptable.

A2.4.3 *Reliability assessment*

Reliability of the two alternative networks has been tested by assuming a single pipe failure and running the simulations at 75% of the demand

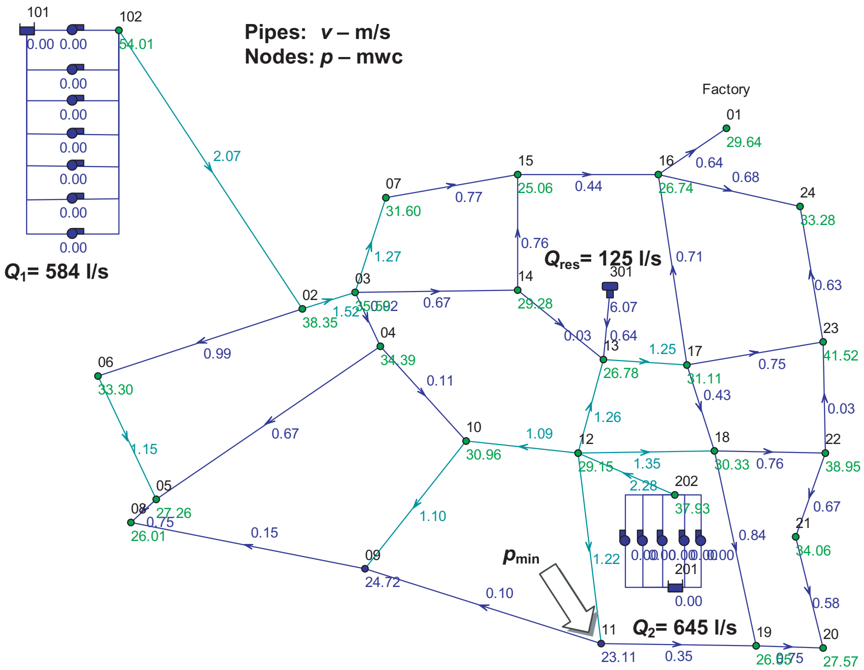


Figure A2.68. S30B3: fire demand remedy 1, operation at 12:00.

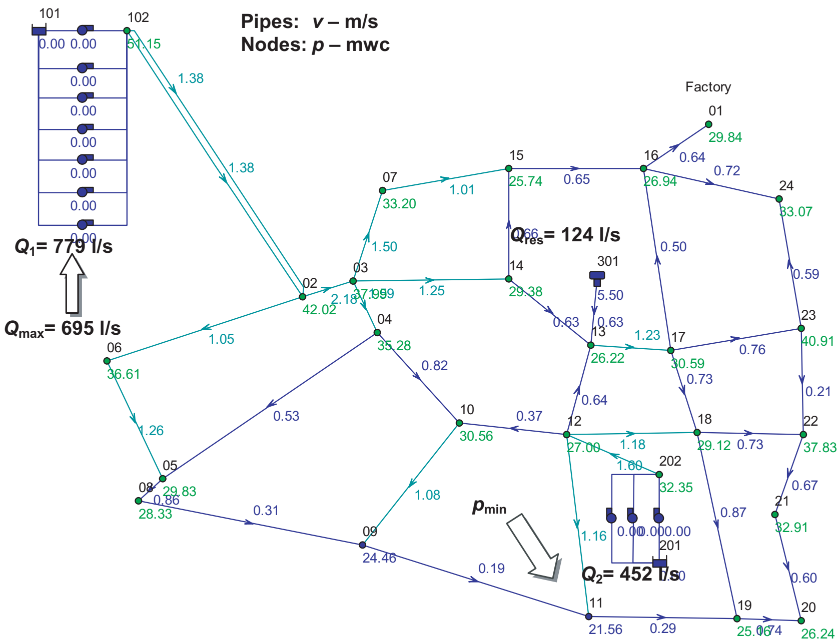


Figure A2.69. S30B3: remedy 2, operation at 12:00.

on the maximum consumption day except for the factory where the demand of 30.09 l/s has remained constant.

Effects of the failure have been analysed for each pipe in the system. The nodes where the pressure drops below 20 mwc during the maximum consumption hour, as a result of the failure, have been identified.

The direct pumping layout from the S30A3 file has been tested, while in the case of Alternative B this was the layout from the S30B3 file. In both cases, the new input files have been created, by dropping the demand (through the demand multiplier) to 75% and switching

Table A2.18. Alternative A: reliability assessment (S30A7).

Pipe burst	Nodes with $p < 20$ mwc	p_{min} (mwc) in node (x)	p_{fact} (mwc)	Q_1 (l/s)	Q_2 (l/s)
02-03[1]	None	39.76 (11)	43.27	440	546
02-06	05, 06, 08	−10.05 (08)	43.43	439	547
03–04	None	34.78 (09)	42.63	415	571
04-05	None	40.22 (11)	43.78	450	536
05-06	05, 08	16.58 (08)	43.55	443	543
05-08	None	27.06 (08)	43.70	447	539
08-09	None	40.20 (11)	43.76	449	537
09-10	09	−37.98 (09)	43.89	452	534
04-10	None	38.06 (09)	43.22	431	555
10-12	None	38.77 (09)	44.10	459	527
09-11	None	40.32 (11)	43.77	449	537
11-12	None	39.19 (11)	43.76	448	538
12-18	None	36.73 (12)	44.05	474	512
11-19	None	22.73 (11)	43.83	454	532
18-19	11, 19, 20	−9.96 (11)	43.86	459	527
19-20	20	19.96 (20)	43.75	449	537
20-21	None	40.13 (11)	43.77	449	537
21-22	21	16.95 (21)	43.79	449	537
18-22	None	34.53 (21)	43.17	451	535
22-23	None	40.22 (11)	43.65	449	537
17-18	None	28.78 (16)	31.82	515	471
12-13	None	40.15 (11)	43.84	449	537
03-14	None	37.33 (15)	41.61	419	567
03-07	07	−393.98 (07)	42.84	443	543
07-15	None	40.20 (11)	43.74	449	537
14-15	15	−83.19 (15)	40.62	450	536
13-14	None	40.28 (11)	43.87	451	535
13-17	None	39.48 (15)	44.55	462	524
17-23	None	26.16 (24)	42.70	448	538
15-16	None	40.20 (11)	43.46	449	537
16-17	01, 16	−200.07 (16)	−197.73	451	535
16-24	None	40.21 (11)	43.65	449	537
23-24	24	−13.79 (24)	41.68	449	537
102-02[1]	None	35.33 (09)	39.46	374	612
202-18[1]	None	35.89 (11)	39.59	503	483

[1] Assumes the burst of one of the pipes in parallel.

the stand-by pumps on. The new file names are S30A7.NET and S30B5.NET, respectively. The calculation results are shown in Tables A2.18 and A2.19.

Clarification:

- The tables indicate the nodes with pressure below 20 mwc, the minimum pressure value, node number and pressure in the factory. Furthermore the supply flows are shown for Source 1, Source 2 and the reservoir (Alternative B).
- The negative value for the reservoir indicates the inflow (refilling).

Table A2.19. Alternative B: reliability assessment (S30B5).

Pipe burst	Nodes with $p < 20$ mwc	p_{min} (mwc) in node (x)	p_{fact} (mwc)	Q_1 (l/s)	Q_2 (l/s)	Q_r (l/s)
02-03	None	23.02 (11)	28.31	175	456	356
02-06	05, 06, 08	−54.44 (08)	31.80	573	441	−27
03-04	None	24.29 (11)	32.00	551	448	−12
04-05	None	25.31 (11)	31.81	589	437	−40
05-06	05, 08	−11.29 (08)	31.80	578	439	−31
05-08	08	−82.22 (08)	31.81	585	438	−37
08-09	None	25.28 (11)	31.80	588	437	−39
09-10	09	−261.73 (09)	31.72	592	435	−41
04-10	None	24.67 (11)	31.87	567	444	−25
10-12	None	25.45 (11)	31.74	601	431	−46
09-11	None	25.21 (11)	31.81	588	437	−39
11-12	11	7.21 (11)	31.43	588	433	−35
12-18	None	22.72 (19)	29.96	585	422	−20
11-19	None	26.24 (11)	31.75	588	436	−38
18-19	19, 20	14.68 (19)	32.04	589	439	−42
19-20	20	3.61 (20)	31.79	588	437	−39
20-21	None	25.20 (11)	31.81	588	437	−39
21-22	21	0.08 (21)	31.84	588	437	−39
18-22	20, 21, 22	−24.24 (21)	31.82	588	437	−39
22-23	None	25.30 (11)	31.81	588	437	−39
17-18	None	24.99 (11)	31.96	588	439	−41
12-13	None	27.04 (13)	32.56	571	408	8
03-14	None	25.94 (11)	32.21	546	430	11
03-07	None	21.31 (15)	26.85	549	435	2
07-15	None	23.85 (15)	29.03	562	436	−12
14-15	None	25.23 (11)	31.61	589	437	−40
13-14	None	25.73 (11)	32.25	566	433	−13
13-17	None	23.36 (11)	27.30	603	446	−63
17-23	23, 24	8.79 (23)	31.68	589	437	−40
15-16	None	24.95 (11)	29.73	577	438	−28
16-17	None	25.33 (11)	31.04	590	437	−41
16-24	None	25.29 (11)	31.84	588	437	−39
23-24	None	25.31 (11)	31.77	588	437	−39
102-02	05, 08, 09, 15	18.77 (08)	24.64	0	521	465
202-12	None	20.72 (11)	29.83	618	0	368

Conclusions:

- Both layouts show fairly good reliability level. The effects of the pipe bursts are for majority of the nodes minimal. In few cases the pressure will be affected in the vicinity of the burst location. The most critical pipes appear to be 02-06, 18-19 and 16-17 in case of the A layout and 02-06, 18-19, 17-23 and 102-02 in case of the B layout.
- The direct pumping alternative shows higher pressures in general, which gives impression that this network has more reserve capacity i.e. more room for reduction of the pipe diameters than the other one.

Table A2.20. Alternative A: reliability assessment for branched secondary mains (S6A7-1).

Pipe burst	Nodes with $p < 20$ mwc	p_{min} (mwc) in node (x)	p_{fact} (mwc)	Q_1 (l/s)	Q_2 (l/s)
02-03	01, 03, 04, 07, 09-20	2.95 (15)	15.81	106	413
02-06	None	22.59 (08)	41.61	274	245
03-04	None	26.57 (11)	33.87	211	308
04-05	None	35.88 (11)	41.93	277	242
05-06	None	32.09 (08)	41.72	275	244
05-08	08	−32.16 (08)	41.81	276	243
08-09	None	35.82 (11)	41.88	277	242
09-10	09	−154.27 (09)	41.94	277	242
04-10	None	28.00 (11)	35.08	220	299
10-12	None	34.12 (11)	40.28	262	257
09-11	None	35.66 (11)	41.87	277	242
11-12	None	25.99 (11)	41.83	276	243
12-18	None	31.92 (13)	43.78	299	220
11-19	None	33.59 (11)	41.90	277	242
18-19	19	19.11 (19)	41.95	278	241
19-20	20	19.22 (20)	41.82	277	242
20-21	None	35.87 (11)	41.87	277	242
21-22	None	35.74 (11)	41.88	277	242
18-22	None	35.66 (11)	41.63	277	242
22-23	None	35.82 (11)	41.82	277	242
17-18	01, 16, 17, 23, 24	−172.42 (16)	−172.39	284	235
12-13	None	24.08 (08)	41.67	277	242
03-14	None	29.21 (15)	41.29	274	245
03-07	None	26.72 (15)	41.31	275	244
07-15	None	34.88 (15)	41.60	276	243
14-15	None	34.85 (15)	41.62	276	243
13-14	None	35.79 (11)	41.86	276	243
13-17	None	35.82 (11)	41.83	277	242
17-23	None	35.80 (11)	41.81	277	242
15-16	None	35.80 (11)	41.77	276	243
16-17	01, 16, 24	−176.49 (16)	−173.46	280	239
16-24	None	26.42 (24)	42.08	277	242
23-24	None	35.81 (11)	41.86	277	242
102-02	01-24	−32.25 (15)	−6.89	0	519
202-18	01-24	−49.86 (11)	−45.29	519	0

This conclusion will be taken into account while deciding the final layout of this alternative.

As a comparison, the initially developed layout with the branched structure of the secondary mains, shown in Figure A2.15 (S6A7), has been tested on reliability as well. The results are shown in Table A2.20.

Conclusions:
- Large parts of network will loose pressure in case pipes 02-03, 17-18, 102-02 and 202-18 burst. The critical pipe is also 16-17. In case the branched layout of the secondary pipes would have to be maintained, laying of the parallels is inevitable in case of the listed pipes.

A2.5 FINAL LAYOUTS

A2.5.1 *Alternative A – Direct pumping*

Beginning of the design period
The following adjustments of the layout proposed in Paragraph 3.6 have been introduced based on the conclusions regarding the system operation:
- The diameters of a few, mainly peripheral pipes have been reduced.
- To improve the supply reliability of the factory:
  - pipe 15-16 has been enlarged to $D = 300$ mm,
  - parallel pipes, $D = 300$ mm, have been laid along route 01–16.
- In order to maintain the system reliability in general, parallel pipes, $D = 500$ mm, have been laid along route 202-18.
- Slightly smaller pump units have been proposed for the pumping station in Source 1.

As in the conclusions of Paragraph 3.6:
- the second source has been immediately connected to the system,
- the storage volume has been immediately constructed for the demand at the end of the design period.

The main characteristics of the pumping stations and the storage volume are given in Table A2.21.

Table A2.21. Alternative A: beginning of the design period.

	Pumping stations			Storage volume		
	Q_p (l/s)	H_p (mwc)	No. of units	D (m)	H (m)	V_{tot} (m^3)
Source 1	100	50	3 + 1	50	5.5	10,800
Source 2	90	40	3 + 1	50	5.5	10,800

Table A2.22. Alternative A: pumping schedule for manual operation (S6A9).

SOURCE 1	Schedule from 0–24	SOURCE 2	Schedule from 0–24
1001	111111111111111111111111	2001	111111111111111111111111
1002	000000011111111111111110	2002	111000111111000111111111
1003	000000000011110000011110	2003	000000001110000011100000
1004[1]	000000000000000000000000	2004[1]	000000000000000000000000

[1] Stand-by units.

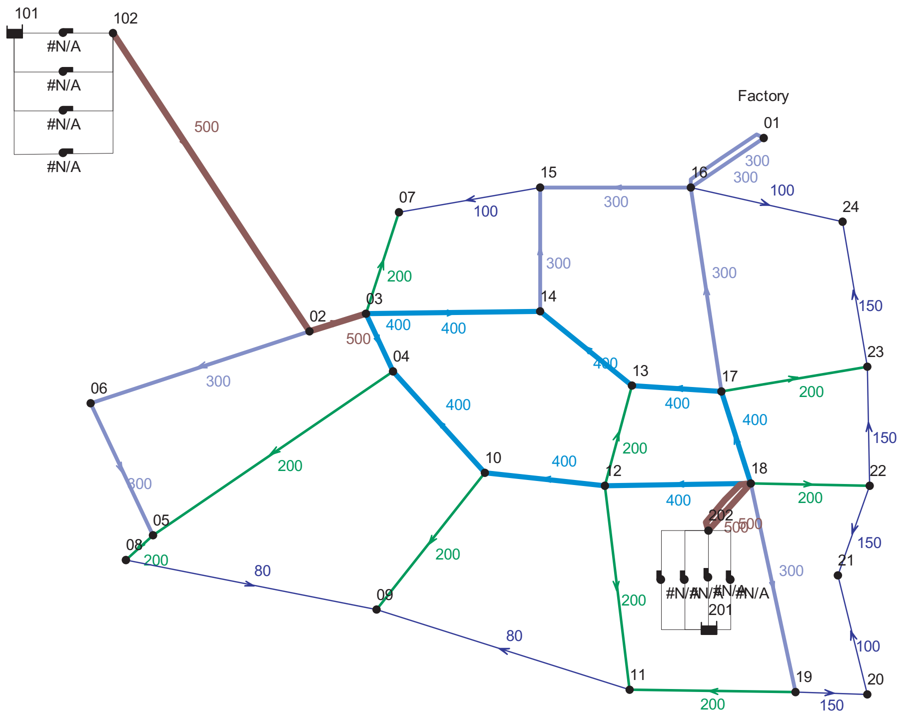


Figure A2.70. Alternative A: pipe diameters at the beginning of the design period.

The final layout of the system at the beginning of the design period is shown in Figure A2.70 (file S6A9.NET). For the manual operation as shown in Table A2.22, the range of pressures in the system on the maximum consumption day is shown in Figure A2.71.

The registered pressure in the factory during the maximum consumption hour is 30.38 mwc. The following pressures are observed in

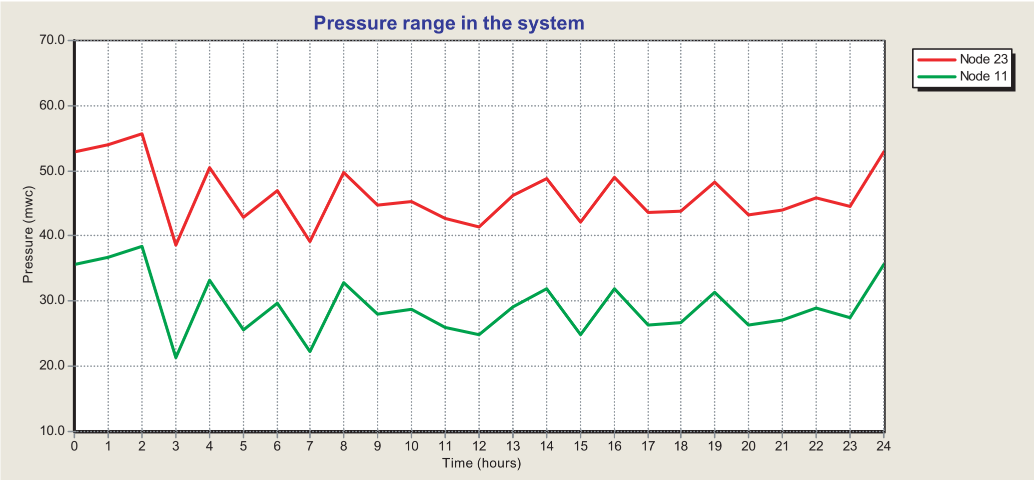


Figure A2.71. S6A9: manual pump control (schedule in Table A2.22).

the case of the pipe burst (the stand-by pumps are Off):

Burst pipe	p_{fact} (mwc)
15-16	28.79
16-17	24.73
16-24	30.53
01-16 (single)	30.18

The pressure during the fire fighting in the factory is 34.12 mwc (the stand-by pumps = On).

The reliability assessment for the 75% demand conditions shows insufficient pressure during the following pipe bursts (the stand-by pumps are On):

Burst pipe	Nodes with $p < 20$ mwc	p_{min} (mwc)	p_{fact} (mwc)
05-08	08	−31.44	48.51
09-10	09	−144.73	48.55
102-02	09, 13 and 15	17.24	23.63

Except for Node 13, all indicated are peripheral nodes. The table shows that the effects of the listed pipe bursts are localised to very small parts of the system and the situation can therefore be considered as satisfactory. The pressure in the factory will also not be affected.

End of the design period

The following system extension is planned to satisfy the demand at the end of the design period (file S30A8.NET):

Pipe	D_{old} (mm)	D_{new} (mm)
102-02	500	2 × 500
02-03	500	2 × 500
02-06	300	400
09-10	200	300
11-19	200	300
18-19	300	400

Three additional pump units of the same size as the existing ones, are planned in each pumping station. For the manual operation as shown in Table A2.23, the range of pressures in the system during the maximum consumption day is shown in Figure A2.72.

Table A2.23. Pumping schedule – Alternative A, manual operation (S30A8).

SOURCE 1	Schedule from 0–24	SOURCE 2	Schedule from 0–24
1001	1111111111111111111111111	2001	1111111111111111111111111
1002	1111111111111111111111111	2002	1111111111111111111111111
1003	1100001111111110001111111	2003	0000001111111111111111110
1004	0000000111111100001111110	2004	0000000111111100001111110
1005	0000000001111000000111110	2005	0000000001111000000011110
1006	0000000000011000000011100	2006	0000000000011000000011100
1007	0000000000000000000000000	2007	0000000000000000000000000

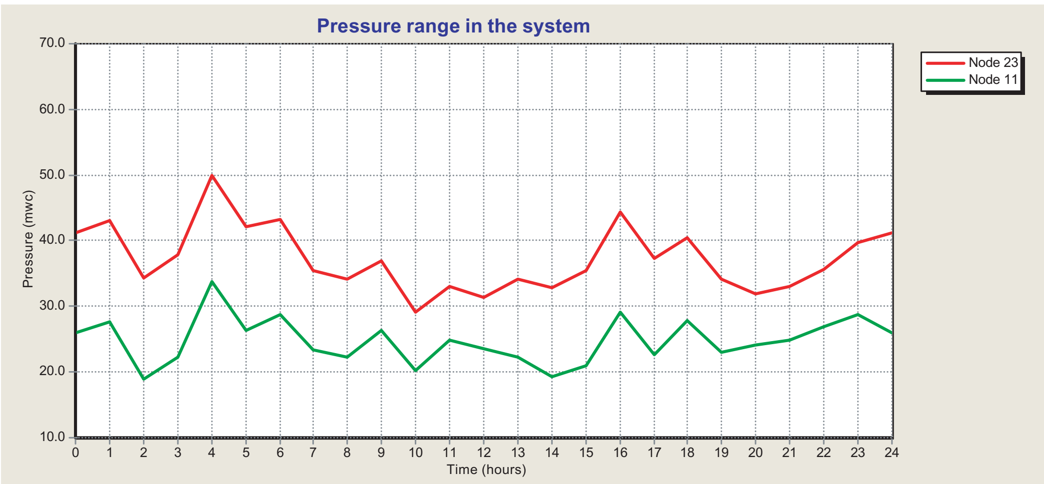


Figure A2.72. S30A8: manual pump control (schedule in Table A2.23).

Pressure in the factory during the maximum consumption hour is 26.15 mwc. The following pressures are observed in the case of the pipe burst:

Burst pipe	Stand-by pumps	p_{fact} (mwc)
15-16	Off	23.80
16-17	On	19.32
16-24	Off	26.61
01-16 (single)	Off	25.95

The pressure during the fire fighting in the factory reaches $p_{fact} = 25.23$ mwc, with the stand-by pumps turned on. This pressure is below the required minimum of 30 mwc and local pressure boosting is suggested.

The reliability assessment for the 75% demand conditions shows insufficient pressure during the following pipe bursts (the stand-by pumps are On) file S30A8-1.NET:

Burst pipe	Nodes with $p < 20$ mwc	p_{min} (mwc)	p_{fact} (mwc)
02-06	05, 06 and 08	−54.97	43.25
05-06	05 and 08	−5.56	43.36
05-08	08	−317.41	43.53
09-10	09	−893.27	43.68
18-19	11, 19 and 20	−18.03	43.01
19-20	20	3.20	43.48
21-22	21	17.26	43.62
18-22	21	19.53	42.93
03-07	07	−395.09	41.12
17-23	24	15.50	43.54
23-24	24	−12.71	42.96

Here as well, the pressure drop is localised to a few peripheral nodes with the factory pressure not being affected by the disaster. Hence, the system reliability can be considered as acceptable.

A2.5.2 *Alternative B – Pumping and balancing storage*

Beginning of the design period

The following adjustments of the layout presented in Paragraph 3.6 have been introduced: a few, mainly peripheral pipes, have been reduced in diameter.

- To improve the supply reliability of the factory, parallel pipes of $D = 300$ mm have been laid along route 01–16.

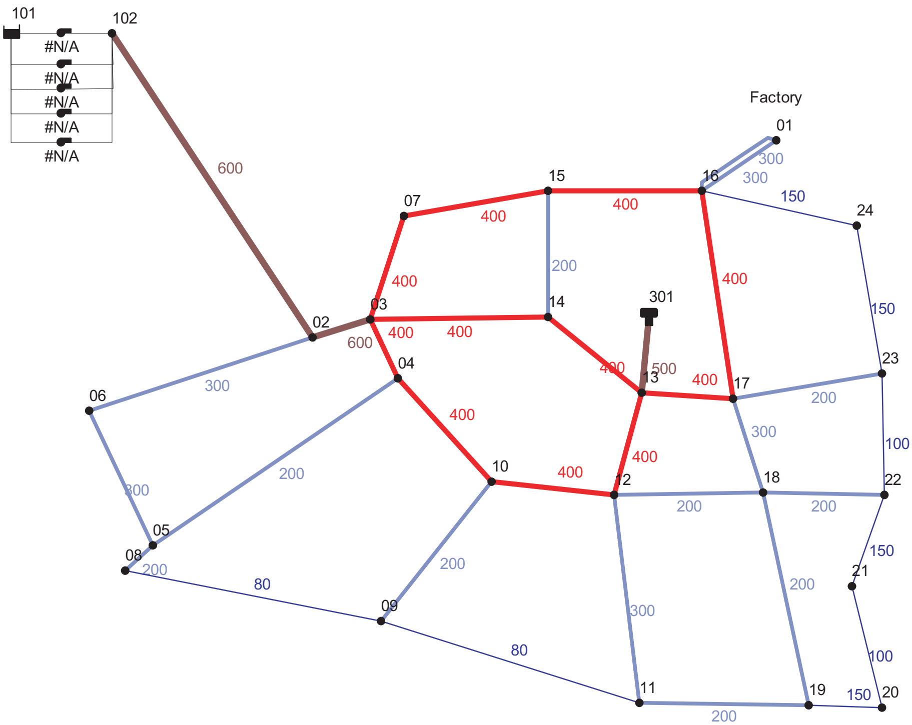


Figure A2.73. Alternative B: pipe diameters at the beginning of the design period (S6B4).

- The balancing tank, $D = 40$ m, $H = 6.5$ m has been adopted. The height of the tank has been set to 20 m.
- Smaller pumping units are proposed in Source 1. One extra unit, out of three planned for the extension, will be installed immediately for more flexible supply in irregular situations.

As in the conclusions of Paragraph 3.6, the storage volume at Source 1 has been immediately constructed for the demand at the end of the design period.

The information about the pumps and the storage volume is given in Table A2.24.

The final layout of the system at the beginning of the design period is shown in Figure A2.73 (file S6B4.NET).

For constant average operation of the pumping station, the range of pressures in the system on the maximum consumption day is shown in Figure A2.74 and the volume variation of the balancing tank in Figure A2.75.

Table A2.24. Alternative B: beginning of the design period.

	Pumping stations			Storage volume		
	Q_p (l/s)	H_p (mwc)	No. of units	D (m)	H (m)	V_{tot} (m^3)
Source 1	130	50	4 + 1	30	6.5	4600
20 m-high tank	—	—	—	40	6.5	8170

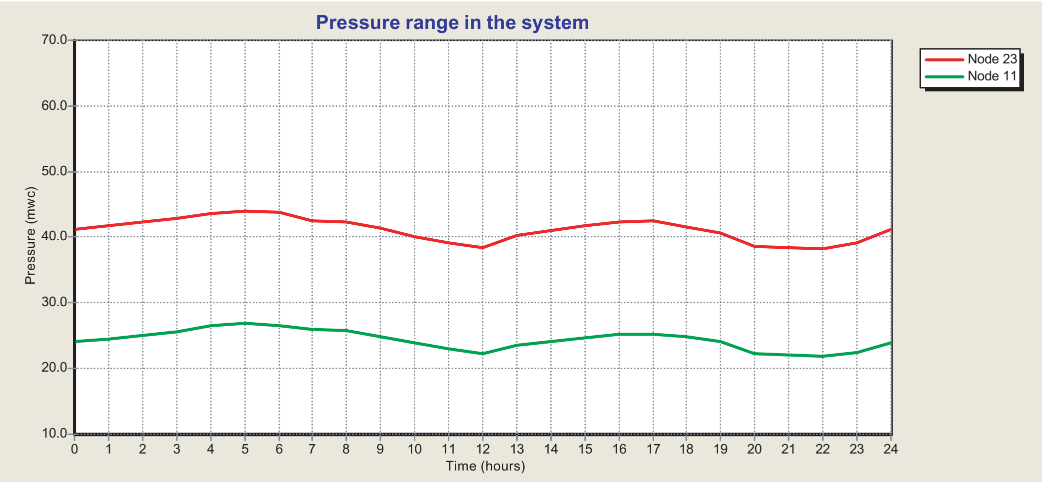


Figure A2.74. S6B4: 20 m-high balancing tank, D = 40 m, H = 6.5 m.

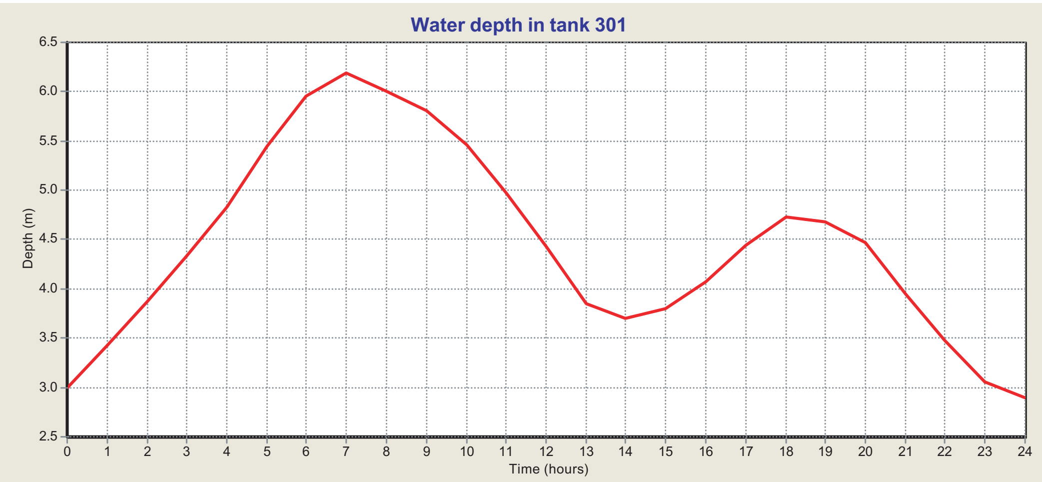


Figure A2.75. S6B4: 20 m-high balancing tank, D = 40 m, H = 6.5 m.

Pressure in the factory during the maximum consumption hour is 29.69 mwc. The following pressures are observed in case of the pipe burst (the stand-by pumps are turned Off):

Burst pipe	p_{fact} (mwc)
15-16	27.17
16-17	29.34
16-24	29.72
01-16 (single)	29.50

The pressure during the fire fighting in the factory is $p_{fact} = 28.59$ mwc (the stand-by pumps are On).

The reliability assessment for the 75% demand conditions shows insufficient pressure during the following pipe bursts (the stand-by pumps are turned on):

Burst pipe	Nodes with $p < 20$ mwc	p_{min} (mwc)	p_{fact} (mwc)
02-06	05, 06 and 08	13.19	32.80
05-08	08	−47.94	32.78
09-10	09	−159.89	32.73
11-12	11 and 19	10.34	32.46

End of the design period
Compared to the summary in Paragraph 3.6, the extent of renovation towards the end of the design period has been reduced. The following reconstruction is planned (file S30B6.NET):

Pipe	D_{old} (mm)	D_{new} (mm)
02-06	300	400
09-10	200	300
11-12	300	400
12-18	200	400
18-19	200	300

- two additional pump units of the same type as the existing ones are going to be installed in the pumping station of Source 1,
- the pumping station in Source 2 is going to be built with three units of the following duty flow and head: $Q_d = 120$ l/s, $H_d = 40$ mwc,
- the clear water reservoir, $V = 4600$ m^3 is going to be constructed in Source 2,
- the diameter of the balancing tank is to be increased from $D = 40$ to 50 m, which effectively means the construction of additional compartment, $D = 30$ m, besides the existing one. The total additional

volume is accidentally equal to those of the clear water reservoirs in the two sources (V = 4600 m^3).

For the average flow supply from the pumping stations, the system operation on the maximum consumption day is shown in Figures A2.76 and A2.77.

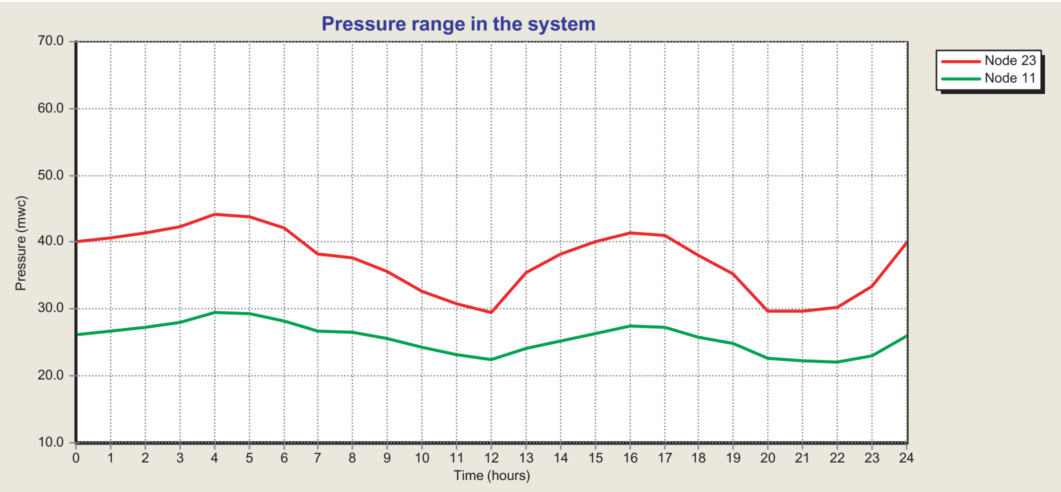


Figure A2.76. S30B6: 20 m-high balancing tank, D = 50 m, H = 6.5 m.

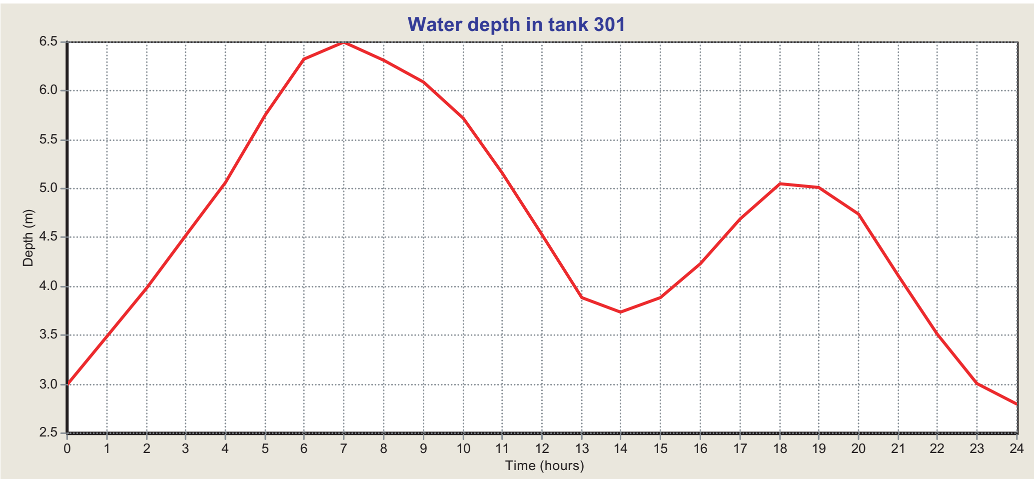


Figure A2.77. S30B6: 20 m-high balancing tank, D = 50 m, H = 6.5 m.

Pressure in the factory during the maximum consumption hour is 27.28 mwc. The following pressures are observed in the case of the pipe burst (the stand-by pumps are Off):

Burst pipe	p_{fact} (mwc)
15-16	24.43
16-17	24.01
16-24	27.49
01-16 (single)	27.09

During the fire fighting in the factory, the pressure on the maximum consumption hour is $p_{fact} = 26.28$ mwc (stand-by pumps are On). This pressure is below the required minimum of 30 mwc and local pressure boosting is advised.

The reliability assessment for the 75% demand conditions (stand-by pumps are On) shows insufficient pressure during the following pipe bursts:

Burst pipe	Nodes with $p < 20$ mwc	p_{min} (mwc)	p_{fact} (mwc)
06	05, 06 and 08	−66.86	32.06
05-06	05 and 08	−17.45	32.06
05-08	08	−329.39	32.06
09-10	09	−904.88	32.03
11-12	11	4.89	31.59
18-19	19	18.72	32.23
19-20	20 and 21	−11.42	32.02
21-22	21	1.04	32.07
18-22	20, 21 and 22	−28.40	32.02
17-23	23 and 24	6.00	31.93
102-02	05, 08, 09 and 15	18.57	24.77

A2.5.3 *Phased development*

Running the simulation of the layouts proposed at the beginning of the design period for the demand conditions at the end of the design period suggests the moment when the system extension should take place. Alternative A:

- With the stand-by pumps switched on, the S6A9 layout will show a considerable drop of pressure for the demands above ±850 l/s (3060 m^3/h). This demand corresponds to the maximum consumption hour demand around Year 15–16 (see Table A2.6). Hence, the proposed year of the system extension is Year 15. As an additional safeguard, five (instead of four) pumps will be installed in both pumping stations at the beginning of the design period, to facilitate the pressure until the extension will take place. The two remaining units are left for the second phase.

Alternative B:

- In the case of layout S6B4, the pressure problems start for the demands around ± 950 l/s (3420 m^3/h), which is the maximum consumption hour demand around Year 19–20. The reason for this is the reservoir itself; it facilitates the pressure as long as it is not empty. To prevent this, the final decision has been made to build the pumping station with the full number of seven units already at the beginning of the design period. This should help maintaining the balancing role of the reservoir in the years before the extension will take place. The proposed year of the system extension is Year 19.

The Tables A2.25 and A2.26 show the summary of the phased development of the system for the two alternatives.

A2.5.4 *Cost analyses*

Based on the prices listed in Tables A2.3 and A2.4, the following cost calculations of the proposed layouts have been conducted.

Table A2.25. Alternative A: phased development.

	Beginning of the design period	Construction at year 15	End of the design period
Pipe diameter (mm)	Total length (m)		
D = 80	2700	—	2700
D = 100	2200	—	2200
D = 150	2550	—	2550
D = 200	6850	—	6850
D = 300	5950	1650	7600
D = 400	4800	2100	6900
D = 500	3500	2000	5500
D = 600	—	—	—
Total pipes (m)	28,550	5750	34,300
Pumping stations	Number of units / Q_p (l/s) / H_p (mwc)		
Source 1	5 / 100 / 50	2 / 100 / 50	7 / 100 / 50
Source 2	5 / 90 / 40	2 / 90 / 40	7 / 90 / 40
Storage	Volume (m^3)		
Source 1	10,800	—	10,800
Source 2	10,800	—	10,800
Total volume (m^3)	21,600	—	21,600

Table A2.26. Alternative B: phased development.

	Beginning of the design period	Construction at year 19	End of the design period
Pipe diameter (mm)	Total length (m)		
D = 80	2700	—	2700
D = 100	1200	—	1200
D = 150	2750	—	2750
D = 200	7250	—	7250
D = 300	4050	2050	6100
D = 400	7100	2950	10,050
D = 500	100	—	100
D = 600	2000	—	2000
Total pipes (m)	27,150	5000	32,150
Pumping stations	Number of units / Q_p (l/s) / H_p (mwc)		
Source 1	7 / 130 / 50	—	7 / 130 / 50
Source 2	—	3 / 120 / 40	3 / 120 / 40
Storage	Volume (m^3)		
Source 1	4600	—	4600
Source 2	—	4600	4600
Water tower	8170	4600	12,770
Total volume (m^3)	12,770	9200	21,970

Investment costs

The investment costs of the final layouts are summarised in Table A2.27 for Alternative A and Table A2.28 for Alternative B.

Clarification:

- The installed capacity of the pumping stations has been assumed as $Q_{max} = 1.5 \times Q_d$, with all units (including stand-by) in operation. Note that the price is calculated for the capacities converted from l/s into m^3/h.
- Investment price of the water tower includes the cost of the supporting structure.

Assuming that 60% of the investment needed at the beginning of the design period will be borrowed immediately, 40% in Year 4 and the entire sum for the second phase in the year when the extension will take place, the total values of the investments brought to Year 6 are (the loan interest is 8%):

Alternative A:

$$0.6 \times 13{,}485{,}481 \div 1.08^{-5} + 0.4 \times 13{,}485{,}481 \div 1.08^{-2} + 4{,}026{,}430 \div 1.08^{9} = \text{US\$ } 20{,}194{,}761.–$$

Table A2.27. Alternative A: investment costs.

	Beginning of the design period		Investment at year 15	
Pipe diameter (mm)	Total length (m)	Total price (US$)	Total length (m)	Total price (US$)
$D = 80$	2700	162,000.–	—	—
$D = 100$	2200	154,000.–	—	—
$D = 150$	2550	229,500.–	—	—
$D = 200$	6850	890,500.–	—	—
$D = 300$	5950	1,071,000.–	1650	297,000.–
$D = 400$	4800	1,248,000.–	2100	546,000.–
$D = 500$	3500	1,085,000.–	2000	620,000.–
$D = 600$	—	—	—	—
Total pipes	28,550	4,840,000.–	5750	1,463,000.–
Pumping stations	Installed capacity (m^3/h)	Total price (US$)	Installed capacity (m^3/h)	Total price (US$)
Source 1	$5 \times 540 = 2700$	2,780,104.–	$2 \times 540 = 1080$	1,335,700.–
Source 2	$5 \times 486 = 2430$	2,555,377.–	$2 \times 486 = 972$	1,227,730.–
Total PST	5130	5,335,481.–	2052	2,563,430.–
Reservoirs	Total volume (m^3)	Total price (US$)	Total volume (m^3)	Total price (US$)
Source 1	10,800	1,655,000.–	—	—
Source 2	10,800	1,655,000.–	—	—
Total reservoirs	21,600	3,310,000.–	—	—
Total A	—	13,485,481.–	—	4,026,430.–

The re-payment of the loan starts at Year 6. The annual instalments to be paid are:

$$20{,}194{,}761 \times (0.08 \times 1.08^{25}) \div (1.08^{25} - 1) = \text{US\$}1{,}891{,}821.–$$

Alternative B:

$$0.6 \times 11{,}726{,}375 \div 1.08^{-5} + 0.4 \times 11{,}726{,}375 \div 1.08^{-2}$$

$$+ 4{,}999{,}603 \div 1.08^{13} = \text{US\$ } 17{,}647{,}336.–$$

The annual instalments to be paid are:

$$17{,}647{,}336 \times (0.08 \times 1.08^{25}) \div (1.08^{25} - 1) = \text{US\$}1{,}653{,}381.–$$

Conclusions:
- Alternative B is cheaper than A. The reason is the lower initial investment and later start of the second phase.

Table A2.28. Alternative B: investment costs.

	Beginning of the design period		Investment at year 19	
Pipe diameter (mm)	Total length (m)	Total price (US$)	Total length (m)	Total price (US$)
$D = 80$	2700	162,000.–	—	—
$D = 100$	1200	84,000.–	—	—
$D = 150$	2750	247,500.–	—	—
$D = 200$	7250	942,500.–	—	—
$D = 300$	4050	729,000.–	2050	369,000.–
$D = 400$	7100	1,846,000.–	2950	767,000.–
$D = 500$	100	31,000.–	—	—
$D = 600$	2000	720,000.–	—	—
Total pipes	27,150	4,762,000.–	5000	1,136,000.–
Pumping stations	Installed capacity (m^3/h)	Total price (US$)	Installed capacity (m^3/h)	Total price (US$)
Source 1	$7 \times 702 = 4914$	4,488,675.–	—	—
Source 2	—	—	$3 \times 648 = 1944$	2,137,603.–
Total PST	4914	4,488,675.–	1944	2,137,603.–
Reservoirs	Total volume (m^3)	Total price (US$)	Total volume (m^3)	Total price (US$)
Source 1	4600	725,000.–	—	—
Source 2	—	—	4600	725,000.–
Water tower	8170	1,750,700.–	4600	1,001,000.–
Total reservoir	12,770	2,475,700.–	9200	1,726,000.–
Total B	—	11,726,375.–	—	4,999,603.–

- The cost of the elevated balancing tank appears not to be the predominant factor. Alternative A has lower investment costs for the storage but more expensive pumping stations.

Reading:

- See notes, Chapter 4, Section 4.1.2: 'Economic aspects'.

Operation and maintenance costs

The operation and maintenance costs are calculated as a percentage of the raw investment costs (Table A2.4). The average value is taken although a trend of increase might be expected towards the end of the design period.

Alternative A:

Distribution system: $0.005 \times (4{,}840{,}000 + 1{,}463{,}000) = $ US$29,735.–

Pumping stations: $0.020 \times (5{,}335{,}481 + 2{,}563{,}430) = $ US$157,978.–

Reservoirs: $0.008 \times 3{,}310{,}000 = $ US$26,480.–

TOTAL: US$214,193.– per year.

Alternative B:
Distribution system: $0.005 \times (4{,}762{,}000 + 1{,}136{,}000) = \text{US\$}29{,}490.–$
Pumping stations: $0.020 \times (4{,}488{,}675 + 2{,}137{,}603) = \text{US\$}132{,}526.–$
Reservoirs: $0.008 \times (2{,}475{,}700 + 1{,}726{,}000) = \text{US\$}33{,}614.–$
TOTAL: US$195,630.– per year.

Water price increase
From Table A2.6, the average water demand/production at Year 6 is 31,977 m^3/d = 11,671,605 m^3/y.

At the end of the design period, the annual production equals $61{,}344 \times 365 = 22{,}390{,}560$ m^3/y.

The total quantity of water supplied during the design period equals 406,381,510 m^3 or on average 16,255,260 m^3 per year.

Due to the loan repayment and the O&M costs, the average water price increase is going to be:
Alternative A:

$$(1{,}891{,}821 + 214{,}193)/16{,}255{,}260 = \text{US\$}0.13 \text{ per m}^3$$

Alternative B:

$$(1{,}653{,}381 + 195{,}630)/16{,}255{,}260 = \text{US\$}0.11 \text{ per m}^3$$

A2.5.5 *Summary and conclusions*

Design and construction
The design layout is cheaper than in case of Alternative A: it requires lower initial investments and a later extension of the system. The distribution network is purposely left with surplus capacity at the beginning of the design period in order to provide a good supply in the final years before the reconstruction of the system takes place.

Construction of the large elevated tank may prove to be a structural- or even an aesthetic problem, bearing in mind the location that is close to the centre of the town. Otherwise, the price does not seem to be a limitation.

Operation and maintenance
Operation of the system is stable, with mild variations in pressures throughout the day. Moreover it is reasonable to expect that the pumping costs in this scheme of supply will be lower than in Alternative A. The maintenance costs are also lower compared to Alternative A although the price difference is small.

Supply factory

Here as well, Alternative B shows better performance than A. The layout provides good pressures in the factory regardless of calamities in the system. The small deficit of pressure during the fire fighting should be overcome by local measures.

Reliability

Both A- and B-layouts are fairly reliable in case of failure in the system.

Overall conclusion

Given all the facts, the final choice in this exercise is Alternative B.

APPENDIX 3

Minor Loss Factors

The general minor loss formula is:

$$h_m = \frac{8\xi}{\pi^2 gD^4}Q^2 = \frac{\xi}{12.1D^4}Q^2$$

where h_m is the minor loss (mwc), Q is pipe flow (m^3/s), ξ is minor loss factor (–), g is gravity, $g = 9.81$ m/s^2 and D is pipe diameter (m). D is the downstream diameter if the cross-section changes, unless stated differently.

A3.1 BENDS AND ELBOWS

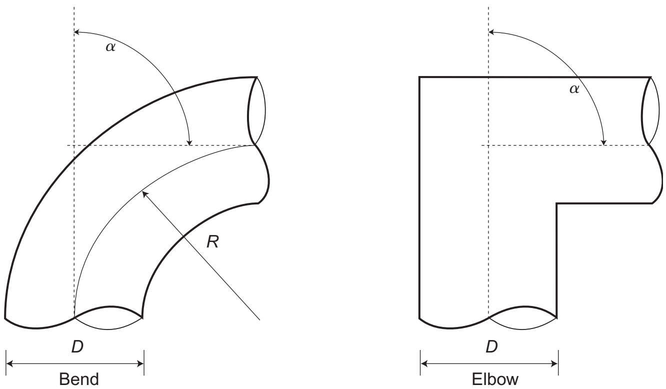


Figure A3.1. Geometry of bends and elbows (Table A3.1).

Table A3.1. Minor loss factor s for elbows and bends (Figure A3.1).

Deflection angle, α (°)	15	30	45	60	90
Bends, $R/D = 1$	0.05	0.09	0.13	0.16	0.21
Bends, $R/D = 2$	0.04	0.07	0.10	0.12	0.15
Bends, $R/D = 3$	0.03	0.05	0.08	0.09	0.12
Bends, $R/D = 4$	0.02	0.04	0.06	0.07	0.09
Elbows	0.06	0.13	0.25	0.50	1.20

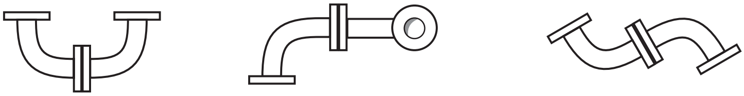


Figure A3.2. Multipliers for coupled 90º-bends.

By coupling two 90°-bends or elbows together, the ξ-value for a single bend/elbow should not be doubled but multiplied by the value shown in Figure A3.2.

A3.2 ENLARGEMENTS AND REDUCERS

The minor loss factor is calculated as:

$$\xi = C\left(\frac{A_2}{A_1} - 1\right)^2$$

where C is the factor dependant on the deflection angle α and $A_{1(2)}$ is the cross-section area up or down-stream of the obstruction (Figure A3.3).

For the same shapes, KSB (1990) recommends the following straightforward ξ-values (see Figure A3.4).

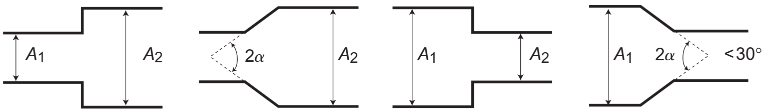


Figure A3.3. Geometry of pipe enlargements and reducers (Table A3.2).

Table A3.2. Minor loss factor C at a gradual change of the cross section.

Deflection angle α (°)	10	20	30	40	50	60	90
C-factor	0.2	0.4	0.7	0.9	1.0	1.1	1.1

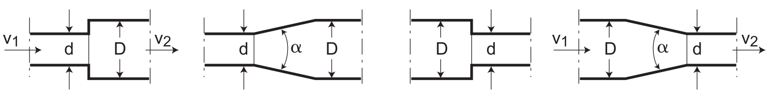


Figure A3.4. Geometry of pipe enlargements and reducers (Table A3.3).

Table A3.3. Minor loss factor ξ, at a gradual change of the cross section (KSB, 1990)

Type $d/D =$	0.5	0.6	0.7	0.8	0.9
Enlarger 1	0.56	0.41	0.26	0.13	0.04
Enlarger 2, $\alpha = 8°$	0.07	0.05	0.03	0.02	0.01
Enlarger 2, $\alpha = 15°$	0.15	0.11	0.07	0.03	0.01
Enlarger 2, $\alpha = 20°$	0.23	0.17	0.11	0.05	0.02
Reducer 3	4.80	2.01	0.88	0.34	0.11
Reducer 4, $20° < \alpha < 40°$	0.21	0.10	0.05	0.02	0.01

A3.3 BRANCHES

The recommended values for ξ are shown in Figure A3.5 and Table A3.4 (KSB, 1990).

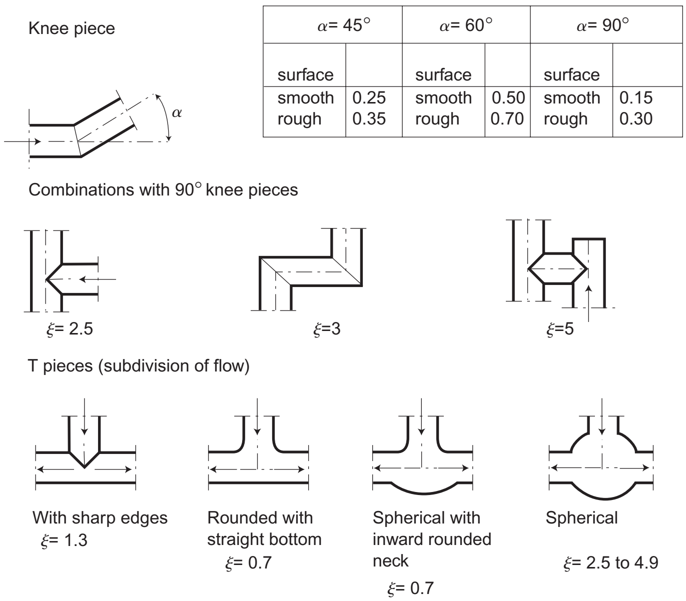


Figure A3.5. Minor loss factors for various types of branches.

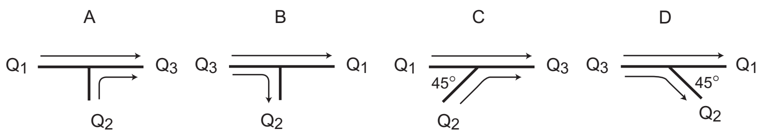


Figure A3.6. Various types of branches (Table A3.4).

Table A3.4. Minor loss factor ξ, at various T-branches (Figure A3.6) (KSB, 1990).

Type	$Q_2/Q_3 =$	0.2	0.4	0.6	0.8	1.0
A	$\xi_2 \approx$	−0.4	0.08	0.47	0.72	0.91
	$\xi_1 \approx$	0.17	0.30	0.41	0.51	—
B	$\xi_2 \approx$	0.88	0.89	0.95	1.10	1.28
	$\xi_1 \approx$	−0.08	−0.05	0.07	0.21	—
C	$\xi_2 \approx$	−0.38	0	0.22	0.37	0.37
	$\xi_1 \approx$	0.17	0.19	0.09	−0.17	—
D	$\xi_2 \approx$	0.68	0.50	0.38	0.35	0.48
	$\xi_1 \approx$	−0.06	−0.04	0.07	0.20	—

A3.4 INLETS AND OUTLETS

The recommended values for ξ are shown in Figure A3.7 (KSB, 1990).

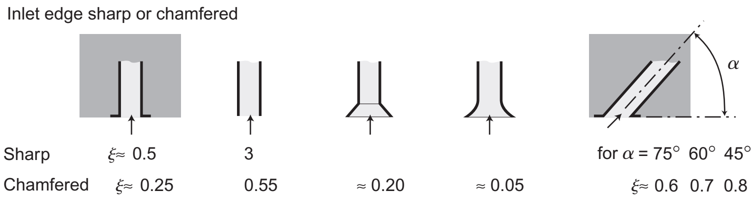


Figure A3.7. Minor loss factors for various types of inlets and outlets.

$\xi = 1$ for downstream of a straight pipe with an approximately uniform velocity distribution in the outlet cross-section.
$\xi = 2$ in the case of a very unequal velocity distribution, e.g. immediately downstream of an elbow or a valve, etc.

A3.5 FLOW METERS

The recommended ξ-values for the Venturi meters and orifice plates are shown in Table A3.5 (Figure A3.8) (KSB, 1990).

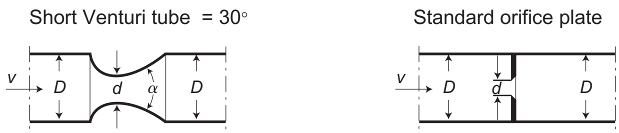


Figure A3.8. Geometry of Venturi tube and standard orifice plate (Table A3.5).

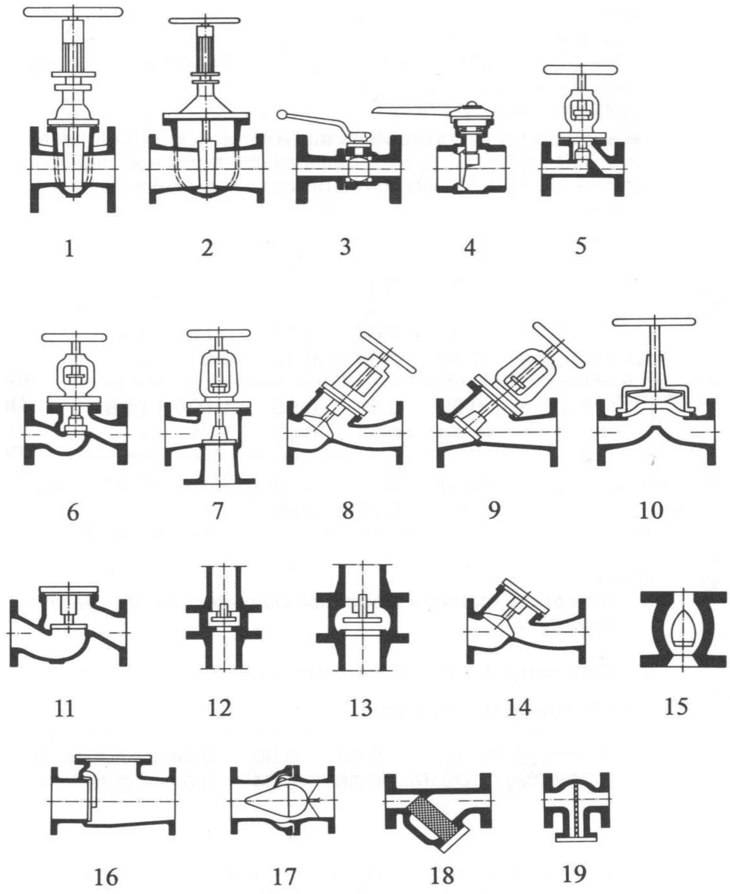
Table A3.5. Minor loss factor ξ, for Venturi tube and orifice plate (Source: KSB, 1990).

Diameter ratio d/D	0.30	0.40	0.50	0.60	0.70	0.80
Aperture ratio $m = (d/D)^2$	0.09	0.16	0.25	0.36	0.49	0.64
Short Venturi tube	21	6	2	0.7	0.3	0.2
Standard orifice plate	300	85	30	12	4.5	2

For volumetric water meters $\xi \approx 10$, as an approximation.

A3.6 VALVES

The recommended ξ-values for various types of valves as shown below are displayed in Figure A3.9 (KSB, 1990).



	Type of valve/fitting		Design[3]	Loss coefficient ξ for DN = 15	20	25	32	40	50	65	80	100	125	150	200	250	300	400	500	600	800	1000	Remarks
Shut-off valves	flat gate valves	min	1	0.1	←	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	→	0.1	for $d_E < DN$ cf. footnote [1])
	(d_E=DN)	max		0.65	0.6	0.55	0.5	0.5	0.45	0.4	0.35	0.3	←	—	—	—	—	—	—	—	→	0.3	
	round-body gate	min	2						0.25	0.24	0.23	0.22	0.21	0.19	0.18	0.17	0.16	0.15	0.13	0.12	0.11	0.11	
	valves (d_E=DN)	max							0.32	0.31	0.30	0.28	0.26	0.25	0.23	0.22	0.20	0.19	0.18	0.16	0.15	0.14	
	cocks (d_E = DN)	min	3	0.10	0.10	0.09	0.09	0.08	0.08	0.07	0.07	0.06	0.05	0.05	0.04	0.03	0.03	0.02					for $d_E < DN$
		max		0.15	←	—	—	—	—	—	—	—	—	—	—	—	→	0.15					ξ = 0.4 to 1.1
	swing-type valves	PN ≧ 2.5	4					0.90	0.76	0.60	0.50	0.42	0.36	0.30	0.25	0.20	0.16	0.13	0.10	0.08	0.06	0.05	
		PN ≦ 40												1.50	1.20	1.00	0.92	0.83	0.76	0.71	0.67	0.63	
	valves, forged	min	5			6.0	←	→	6.0														
		max				6.8	←	→	6.8														
	valves, cast	min	6	3.0	←	—	—	—	—	—	—	—	—	—	—	→	3.0						ξ = 2 to 3 possible for
		max		6.0	←	—	—	—	—	—	—	—	—	—	—	→	6.0						optimized valve
	angle valves	min	7	2.0	←	—	—	—	—	—	—	—	—	—	—	—	→	2.0					
		max		3.1	←	→	3.1	3.4	3.8	4.1	4.4	4.7	5.0	5.3	5.7	6.0	6.3	6.6					
	slanted-seat valves	min	8	1.5	←	—	—	—	—	—	—	—	—	—	—	→	1.5						
		max		2.6	←	—	—	—	—	—	—	—	—	—	—	→	2.6						
	full-bore valves	min	9	0.6	←	—	—	—	—	—	—	—	—	—	—	—	→	0.6					
		max		1.6	←	—	—	—	—	—	—	—	—	—	—	—	→	1.6					
	diaphragm valves	min	10	0.8	←	—	—	—	—	—	—	—	—	→	0.8								
		max		2.2	←	—	—	—	—	—	—	—	—	→	2.2								
Backflow preventors	non-return valves,	min	11	3.0	←	—	—	—	—	—	—	—	—	→	3.0								
	straight-seat	max		6.0	←	—	—	—	—	—	—	—	—	→	6.0								
	non-return valves,	min	12	3.2	←	—	—	→	3.2	3.7	5.0	7.3											
	axial	max		3.4	3.4	3.5	3.6	3.8	4.2	5.0	6.4	8.2											
	non-return valves,	min	13										4.3	←	—	→	4.3						
	axially expanded	max											4.6	←	—	→	4.6						
	non-return valves,	min	14	2.5	2.4	2.2	2.1	2.0	1.9	1.7	1.6	1.5	←	—	—	→	1.5						
	slanted seat	max		3.0	←	—	—	—	—	—	—	—	—	—	—	→	3.0						
	foot valves	min	15						1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.4	0.4	(7.0)	(6.1)	(5.5)	(4.5)	(4.0)	( ) in groups
		max							3.0	←	—	—	—	—	—	→	3.0						
	swing-type check	min	16	0.5	←	—	→	0.5	0.4	←	—	—	—	—	—	→	0.4	0.3	←	—	→	0.3	swing-type valves with-
	valves	max		2.4	2.3	2.3	2.2	2.1	2.0	1.9	1.8	1.8	1.7	1.6	1.5	1.5	1.4	1.3	1.2	1.2	1.1	1.0	out levers and weights [2])
	hydrostops	v = 4 m/s	17						0.9			3.0		3.0	2.5	2.5	1.2	2.2					
		v = 3 m/s							1.8			4.0		4.5	4.0	4.0	1.8	3.4					
		v = 2 m/s							5.0			6.0		8.0	7.5	6.5	6.0	7.0					
	filters		18					2.8	←	—	—	—	—	—	—	→	2.8						in clean condition
	screens		19					1.0	←	—	—	—	—	—	—	→	1.0						

[1]) If the narrowest shut-off diameter d_E is smaller than the nominal diameter DN, the loss coefficient ξ must be increased by $(DN/d_E)^x$, with x = 5 to 6

[2]) In the case of partial opening, i.e. low flow velocities, the loss coefficients increase

Figure A3.9. Minor loss factors for various types of values.

APPENDIX 4

Hydraulic tables (Darcy–Weisbach/Colebrook–White)

k(mm) = 0.01 T(°C) = 10

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.11	0.2	0.8	0.17	0.3	1.2	0.25	0.5	1.8	0.31	0.6	2.2
80	0.16	0.8	2.9	0.24	1.2	4.3	0.35	1.8	6.3	0.44	2.2	7.9
100	0.19	1.5	5.3	0.28	2.2	7.8	0.41	3.2	11.5	0.51	4.0	14.5
125	0.22	2.7	9.6	0.32	4.0	14.3	0.48	5.8	21.1	0.60	7.3	26.4
150	0.25	4.4	15.8	0.37	6.5	23.3	0.54	9.5	34.3	0.68	12.0	43.1
200	0.30	9.5	34.2	0.45	14.0	50.4	0.66	20.6	74.2	0.82	25.8	92.9
250	0.35	17.3	62.3	0.52	25.5	91.7	0.76	37.4	134.6	0.95	46.7	168.3
300	0.40	28.2	101.5	0.59	41.4	149.1	0.86	60.7	218.6	1.07	75.9	273.2
350	0.44	42.6	153.3	0.65	62.5	224.9	0.95	91.5	329.4	1.19	114.3	411.3
400	0.48	60.8	218.8	0.71	89.1	320.9	1.04	130.4	469.5	1.30	162.8	585.9
450	0.52	83.2	299.5	0.77	121.9	438.8	1.12	178.2	641.5	1.40	222.3	800.3
500	0.56	110.1	396.5	0.82	161.2	580.4	1.20	235.5	848.0	1.50	293.7	1057.5
600	0.63	178.8	643.6	0.92	261.4	941.1	1.35	381.5	1373.4	1.68	475.4	1711.6
700	0.70	269.1	968.8	1.02	393.1	1415.2	1.49	573.2	2063.4	1.86	714.0	2570.2
800	0.76	383.3	1380.0	1.11	559.5	2014.4	1.62	815.2	2934.7	2.02	1014.9	3653.8
900	0.82	523.5	1884.7	1.20	763.7	2749.3	1.75	1111.9	4002.7	2.18	1383.8	4981.5
1000	0.88	691.7	2490.2	1.28	1008.4	3630.3	1.87	1467.3	5282.3	2.32	1825.5	6571.8
1100	0.94	889.8	3203.2	1.36	1296.5	4667.3	1.98	1885.4	6787.6	2.47	2345.0	8441.9
1200	0.99	1119.5	4030.3	1.44	1630.5	5869.7	2.10	2370.0	8532.1	2.61	2946.9	10,608.8
1300	1.04	1382.7	4977.9	1.52	2012.9	7246.6	2.20	2924.7	10,529.1	2.74	3635.7	13,088.5
1400	1.09	1681.1	6052.0	1.59	2446.3	8806.8	2.31	3553.0	12,791.0	2.87	4415.7	15,896.5
1500	1.14	2016.3	7258.6	1.66	2933.0	10,558.8	2.41	4258.3	15,330.0	2.99	5291.1	19,048.0
1600	1.19	2389.8	8603.4	1.73	3475.2	12,510.8	2.51	5043.9	18,158.1	3.12	6266.0	22,557.6

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.37	0.7	2.6	0.42	0.8	3.0	0.47	0.9	3.3	0.51	1.0	3.6
80	0.52	2.6	9.3	0.59	2.9	10.6	0.65	3.3	11.7	0.71	3.6	12.8
100	0.60	4.7	17.0	0.68	5.4	19.3	0.76	5.9	21.4	0.82	6.5	23.3
125	0.70	8.6	31.0	0.79	9.8	35.1	0.88	10.8	38.8	0.96	11.8	42.3
150	0.79	14.0	50.5	0.90	15.9	57.2	0.99	17.6	63.2	1.08	19.1	68.8
200	0.96	30.2	108.8	1.09	34.2	123.1	1.20	37.8	136.0	1.31	41.1	148.0
250	1.12	54.7	197.1	1.26	61.9	222.7	1.39	68.4	246.1	1.52	74.4	267.8
300	1.26	88.8	319.8	1.42	100.4	361.3	1.57	110.9	399.1	1.71	120.6	434.1
350	1.39	133.7	481.3	1.57	151.0	543.5	1.73	166.7	600.2	1.88	181.3	652.7
400	1.52	190.4	685.4	1.71	215.0	773.8	1.89	237.3	854.3	2.05	258.0	928.8
450	1.63	260.0	935.9	1.85	293.4	1056.4	2.04	323.9	1166.0	2.21	352.1	1267.4
500	1.75	343.4	1236.3	1.97	387.5	1395.1	2.18	427.7	1539.7	2.37	464.8	1673.3
600	1.96	555.6	2000.1	2.22	626.7	2256.3	2.45	691.5	2489.3	2.66	751.3	2704.7
700	2.17	834.0	3002.2	2.44	940.5	3385.8	2.70	1037.4	3734.7	2.93	1126.9	4057.0
800	2.36	1185.2	4266.5	2.66	1336.2	4810.5	2.93	1473.6	5305.1	3.18	1600.5	5761.9
900	2.54	1615.4	5815.4	2.86	1820.9	6555.3	3.16	2007.8	7228.0	3.43	2180.4	7849.3
1000	2.71	2130.5	7669.9	3.06	2401.2	8644.2	3.37	2647.2	9529.8	3.66	2874.3	10,347.6
1100	2.88	2736.2	9850.4	3.24	3083.3	11,099.9	3.58	3398.7	12,235.3	3.88	3689.9	13,283.7
1200	3.04	3437.9	12,376.4	3.42	3873.4	13,944.1	3.77	4269.0	15,368.6	4.10	4634.4	16,683.7
1300	3.19	4240.7	15,266.5	3.60	4777.2	17,197.9	3.97	5264.6	18,952.6	4.31	5714.6	20,572.4
1400	3.35	5149.7	18,538.8	3.77	5800.4	20,881.5	4.15	6391.6	23,009.6	4.51	6937.2	24,974.0
1500	3.49	6169.6	22,210.7	3.93	6948.5	25,014.5	4.33	7655.9	27,561.2	4.70	8308.8	29,911.7
1600	3.63	7305.4	26,299.3	4.09	8226.7	29,616.0	4.51	9063.4	32,628.2	4.89	9835.6	35,408.2

k(mm) = 0.01 T(°C) = 10

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.55	1.1	3.9	0.59	1.2	4.1	0.62	1.2	4.4	0.69	1.4	4.9
80	0.76	3.8	13.8	0.81	4.1	14.7	0.86	4.3	15.6	0.95	4.8	17.3
100	0.89	7.0	25.1	0.95	7.4	26.8	1.00	7.9	28.4	1.11	8.7	31.4
125	1.03	12.7	45.5	1.10	13.5	48.6	1.17	14.3	51.5	1.29	15.8	57.0
150	1.16	20.6	74.1	1.24	22.0	79.0	1.32	23.3	83.8	1.46	25.7	92.6
200	1.41	44.2	159.3	1.50	47.2	169.9	1.59	50.0	180.0	1.76	55.2	198.8
250	1.63	80.0	288.0	1.74	85.3	307.1	1.84	90.3	325.2	2.03	99.8	359.1
300	1.83	129.7	466.8	1.96	138.2	497.6	2.07	146.4	526.9	2.29	161.6	581.6
350	2.03	194.9	701.7	2.16	207.8	747.9	2.29	219.9	791.8	2.52	242.7	873.8
400	2.21	277.3	998.4	2.35	295.6	1064.0	2.49	312.9	1126.3	2.75	345.2	1242.7
450	2.38	378.4	1362.2	2.54	403.2	1451.6	2.68	426.8	1536.4	2.96	470.8	1694.8
500	2.54	499.5	1798.3	2.71	532.2	1916.0	2.87	563.3	2027.8	3.16	621.2	2236.4
600	2.86	807.2	2906.0	3.04	859.9	3095.8	3.22	909.9	3275.8	3.55	1003.3	3611.8
700	3.15	1210.6	4358.2	3.35	1289.5	4642.0	3.54	1364.2	4911.3	3.91	1503.8	5413.8
800	3.42	1719.1	6188.8	3.64	1830.8	6590.9	3.85	1936.8	6972.4	4.25	2134.5	7684.2
900	3.68	2341.6	8429.7	3.92	2493.5	8976.5	4.15	2637.5	9495.0	4.57	2906.3	10,462.6
1000	3.93	3086.5	11,111.4	4.18	3286.4	11,831.0	4.43	3475.9	12,513.3	4.88	3829.5	13,786.2
1100	4.17	3961.9	14,262.9	4.44	4218.1	15,185.1	4.69	4461.0	16,059.5	5.17	4914.1	17,690.8
1200	4.40	4975.5	17,911.8	4.68	5296.8	19,068.5	4.95	5601.4	20,165.0	5.46	6169.6	22,210.6
1300	4.62	6134.7	22,085.0	4.92	6530.4	23,509.4	5.20	6905.5	24,859.7	5.73	7605.1	27,378.3
1400	4.84	7446.7	26,808.1	5.15	7926.4	28,535.2	5.44	8381.2	30,172.3	6.00	9229.4	33,225.8
1500	5.05	8918.4	32,106.2	5.37	9492.3	34,172.4	5.68	10,036.4	36,131.0	6.25	11,051.0	39,783.6
1600	5.25	10,556.5	38,003.4	5.59	11235.2	40,446.8	5.91	11,878.6	42,762.8	6.50	13,078.2	47,081.7

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.75	1.5	5.3	0.81	1.6	5.7	0.87	1.7	6.1	0.92	1.8	6.5
80	1.04	5.2	18.8	1.12	5.6	20.3	1.20	6.0	21.6	1.27	6.4	22.9
100	1.21	9.5	34.2	1.30	10.2	36.8	1.39	10.9	39.3	1.47	11.6	41.6
125	1.40	17.2	62.0	1.51	18.5	66.7	1.61	19.8	71.2	1.71	20.9	75.4
150	1.58	28.0	100.7	1.70	30.1	108.4	1.82	32.1	115.6	1.92	34.0	122.4
200	1.91	60.1	216.2	2.06	64.6	232.5	2.19	68.9	247.9	2.32	72.9	262.5
250	2.21	108.5	390.5	2.38	116.6	419.8	2.53	124.3	447.4	2.68	131.6	473.6
300	2.48	175.6	632.2	2.67	188.7	679.5	2.85	201.1	724.1	3.01	212.9	766.4
350	2.74	263.8	949.6	2.95	283.5	1020.4	3.14	302.0	1087.2	3.32	319.6	1150.5
400	2.98	375.1	1350.2	3.21	403.0	1450.7	3.42	429.3	1545.4	3.61	454.2	1635.2
450	3.22	511.4	1841.1	3.45	549.4	1977.9	3.68	585.2	2106.7	3.89	619.1	2228.9
500	3.44	674.8	2429.2	3.69	724.8	2609.3	3.93	771.9	2778.9	4.16	816.6	2939.8
600	3.85	1089.5	3922.1	4.14	1170.0	4212.0	4.41	1245.8	4485.0	4.66	1317.8	4743.9
700	4.24	1632.7	5877.7	4.56	1753.1	6311.0	4.85	1866.4	6719.1	5.13	1973.9	7105.9
800	4.61	2317.0	8341.3	4.95	2487.5	8954.9	5.27	2648.0	9532.6	5.57	2800.1	10,080.3
900	4.96	3154.3	11,355.6	5.32	3385.9	12,189.3	5.67	3604.0	12,974.2	5.99	3810.6	13,718.2
1000	5.29	4155.8	14,961.0	5.68	4460.4	16,057.5	6.04	4747.2	17,089.9	6.39	5019.0	18,068.3
1100	5.61	5332.2	19,196.0	6.02	5722.5	20,600.9	6.41	6089.8	21,923.4	6.77	6438.0	23,176.7
1200	5.92	6693.8	24,097.8	6.35	7183.1	25,859.0	6.76	7643.6	27,516.9	7.14	8079.9	29,087.8
1300	6.22	8250.5	29,701.8	6.67	8852.8	31,869.9	7.10	9419.6	33,910.6	7.50	9956.8	35,844.3
1400	6.50	10,011.7	36,042.3	6.98	10,741.8	38,670.4	7.42	11,428.8	41,143.8	7.85	12,079.8	43,487.3
1500	6.78	11,986.8	43,152.6	7.28	12,860.0	46,295.8	7.74	13,681.7	49,254.0	8.18	14,460.2	52,056.6
1600	7.05	14,184.7	51,064.8	7.57	15,216.9	54,780.8	8.05	16,188.3	58,277.8	8.51	17,108.5	61,590.8

k(mm) = 0.05 T(°C) = 10

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.11	0.2	0.8	0.16	0.3	1.2	0.24	0.5	1.7	0.31	0.6	2.2
80	0.16	0.8	2.8	0.23	1.2	4.2	0.34	1.7	6.2	0.43	2.1	7.7
100	0.18	1.4	5.2	0.27	2.1	7.7	0.40	3.1	11.3	0.50	3.9	14.1
125	0.21	2.6	9.5	0.32	3.9	14.0	0.46	5.7	20.5	0.58	7.1	25.6
150	0.24	4.3	15.5	0.36	6.3	22.8	0.53	9.3	33.4	0.66	11.6	41.7
200	0.30	9.3	33.7	0.44	13.7	49.3	0.64	20.0	72.1	0.79	25.0	89.8
250	0.35	17.0	61.2	0.51	24.9	89.5	0.74	36.3	130.6	0.92	45.1	162.5
300	0.39	27.7	99.6	0.57	40.4	145.5	0.83	58.8	211.9	1.04	73.2	263.5
350	0.43	41.7	150.3	0.63	60.9	219.2	0.92	88.5	318.8	1.14	110.0	396.1
400	0.47	59.6	214.4	0.69	86.8	312.5	1.00	126.1	453.8	1.25	156.6	563.6
450	0.51	81.5	293.3	0.75	118.6	427.0	1.08	172.1	619.6	1.34	213.6	769.0
500	0.55	107.8	388.0	0.80	156.8	564.4	1.16	227.3	818.2	1.44	282.0	1015.2
600	0.62	174.8	629.1	0.90	253.9	913.9	1.30	367.5	1323.2	1.61	455.7	1640.4
700	0.68	262.8	946.1	0.99	381.3	1372.8	1.43	551.5	1985.3	1.78	683.3	2459.7
800	0.74	374.0	1346.5	1.08	542.2	1951.9	1.56	783.4	2820.2	1.93	970.1	3492.3
900	0.80	510.4	1837.6	1.16	739.3	2661.5	1.68	1067.3	3842.4	2.08	1321.1	4756.1
1000	0.86	673.9	2426.2	1.24	975.4	3511.4	1.79	1407.2	5065.9	2.22	1741.1	6268.0
1100	0.91	866.3	3118.7	1.32	1253.0	4510.9	1.90	1806.6	6503.8	2.35	2234.5	8044.3
1200	0.96	1089.3	3921.6	1.39	1574.7	5668.8	2.01	2269.1	8168.9	2.48	2805.7	10,100.7
1300	1.01	1344.7	4840.8	1.46	1942.7	6993.8	2.11	2798.1	10,073.2	2.61	3458.9	12,451.9
1400	1.06	1633.9	5882.1	1.53	2359.5	8494.2	2.21	3396.8	12,228.6	2.73	4197.9	15,112.6
1500	1.11	1958.7	7051.2	1.60	2827.2	10,178.0	2.30	4068.5	14,646.5	2.84	5026.8	18,096.5
1600	1.15	2320.4	8353.5	1.67	3348.0	12,052.9	2.40	4816.1	17,337.9	2.96	5949.3	21,417.4

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.36	0.7	2.6	0.41	0.8	2.9	0.45	0.9	3.2	0.49	1.0	3.5
80	0.50	2.5	9.1	0.57	2.8	10.3	0.63	3.1	11.3	0.68	3.4	12.3
100	0.58	4.6	16.5	0.66	5.2	18.7	0.73	5.7	20.6	0.79	6.2	22.4
125	0.68	8.3	30.0	0.77	9.4	33.9	0.85	10.4	37.4	0.92	11.3	40.7
150	0.77	13.6	48.8	0.87	15.3	55.1	0.96	16.9	60.8	1.04	18.3	66.0
200	0.93	29.1	104.9	1.05	32.9	118.3	1.15	36.2	130.5	1.25	39.4	141.7
250	1.07	52.7	189.7	1.21	59.4	213.7	1.33	65.4	235.5	1.45	71.0	255.7
300	1.21	85.4	307.3	1.36	96.1	346.1	1.50	105.9	381.3	1.63	114.9	413.8
350	1.33	128.3	461.8	1.50	144.4	519.9	1.65	159.1	572.6	1.79	172.6	621.2
400	1.45	182.4	656.8	1.63	205.3	739.2	1.80	226.1	814.0	1.95	245.2	882.9
450	1.56	248.8	895.8	1.76	280.0	1008.0	1.94	308.2	1109.7	2.10	334.3	1203.4
500	1.67	328.4	1182.2	1.88	369.4	1329.9	2.07	406.6	1463.7	2.25	440.9	1587.1
600	1.88	530.3	1909.2	2.11	596.4	2146.9	2.32	656.2	2362.3	2.52	711.3	2560.7
700	2.07	794.9	2861.7	2.32	893.6	3216.9	2.55	983.0	3538.7	2.77	1065.3	3835.1
800	2.24	1128.2	4061.5	2.52	1267.9	4564.4	2.77	1394.4	5020.0	3.01	1511.0	5439.6
900	2.41	1536.0	5529.5	2.71	1725.8	6212.8	2.98	1897.7	6831.7	3.23	2056.0	7401.6
1000	2.58	2023.7	7285.4	2.89	2273.3	8184.1	3.18	2499.4	8997.9	3.45	2707.6	9747.3
1100	2.73	2596.7	9347.9	3.07	2916.4	10,499.2	3.37	3206.0	11,541.6	3.65	3472.6	12,501.5
1200	2.88	3259.7	11,735.1	3.24	3660.6	13,178.2	3.56	4023.6	14,485.0	3.85	4357.8	15,688.0
1300	3.03	4017.8	14,464.1	3.40	4511.3	16,240.6	3.74	4958.1	17,849.0	4.05	5369.4	19,329.8
1400	3.17	4875.5	17,551.7	3.56	5473.6	19,704.9	3.91	6015.1	21,654.4	4.23	6513.6	23,448.9
1500	3.30	5837.2	21,014.0	3.71	6552.6	23,589.3	4.07	7200.2	25,920.7	4.41	7796.3	28,066.7
1600	3.44	6907.4	24,866.7	3.86	7753.1	27,911.3	4.24	8518.7	30,667.3	4.59	9223.3	33,204.0

k(mm) = 0.05 T(°C) = 10

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m³/h)	v(m/s)	Q(l/s)	Q(m³/h)	v(m/s)	Q(l/s)	Q(m³/h)	v(m/s)	Q(l/s)	Q(m³/h)
50	0.53	1.0	3.8	0.57	1.1	4.0	0.60	1.2	4.2	0.66	1.3	4.7
80	0.73	3.7	13.3	0.78	3.9	14.2	0.83	4.2	15.0	0.91	4.6	16.5
100	0.85	6.7	24.1	0.91	7.1	25.7	0.96	7.6	27.2	1.06	8.3	30.0
125	0.99	12.1	43.7	1.05	12.9	46.5	1.12	13.7	49.3	1.23	15.1	54.3
150	1.12	19.7	70.9	1.19	21.0	75.6	1.26	22.2	80.0	1.39	24.5	88.1
200	1.35	42.3	152.1	1.43	45.0	162.0	1.51	47.6	171.3	1.67	52.4	188.7
250	1.55	76.2	274.5	1.65	81.2	292.2	1.75	85.8	308.9	1.92	94.5	340.1
300	1.75	123.4	444.1	1.86	131.3	472.6	1.96	138.8	499.6	2.16	152.7	549.9
350	1.92	185.1	666.5	2.05	197.0	709.2	2.16	208.2	749.6	2.38	229.1	824.8
400	2.09	263.1	947.1	2.23	279.9	1007.6	2.35	295.8	1064.8	2.59	325.4	1171.4
450	2.25	358.5	1290.8	2.40	381.4	1373.0	2.53	403.0	1450.8	2.79	443.3	1595.7
500	2.41	472.8	1702.1	2.56	502.9	1810.3	2.71	531.3	1912.7	2.98	584.3	2103.5
600	2.70	762.7	2745.7	2.87	811.0	2919.6	3.03	856.7	3084.3	3.33	941.9	3390.9
700	2.97	1142.1	4111.4	3.16	1214.2	4371.2	3.33	1282.5	4617.1	3.66	1409.7	5075.0
800	3.22	1619.6	5830.6	3.43	1721.7	6198.2	3.62	1818.4	6546.2	3.98	1998.3	7194.0
900	3.46	2203.5	7932.7	3.68	2342.2	8431.9	3.89	2473.5	8904.4	4.27	2717.8	9784.1
1000	3.69	2901.5	10,445.5	3.93	3083.9	11,101.9	4.15	3256.4	11,723.1	4.56	3577.6	12,879.4
1100	3.92	3721.0	13,395.8	4.16	3954.5	14,236.3	4.39	4175.5	15,031.8	4.83	4586.8	16,512.5
1200	4.13	4669.1	16,808.8	4.39	4961.7	17,862.2	4.63	5238.7	18,859.2	5.09	5754.1	20,714.7
1300	4.33	5752.6	20,709.2	4.61	6112.7	22,005.6	4.86	6453.5	23,232.5	5.34	7087.8	25,515.9
1400	4.53	6977.9	25,120.5	4.82	7414.3	26,691.6	5.08	7827.3	28,178.3	5.58	8595.9	30,945.2
1500	4.73	8351.6	30,065.7	5.02	8873.4	31,944.4	5.30	9367.3	33,722.1	5.82	10,286.3	37,030.5
1600	4.91	9879.7	35,566.9	5.22	10,496.5	37,787.5	5.51	11,080.2	39,888.7	6.05	12,166.4	43,799.1

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m³/h)	v(m/s)	Q(l/s)	Q(m³/h)	v(m/s)	Q(l/s)	Q(m³/h)	v(m/s)	Q(l/s)	Q(m³/h)
50	0.72	1.4	5.1	0.78	1.5	5.5	0.83	1.6	5.9	0.88	1.7	6.2
80	0.99	5.0	18.0	1.07	5.4	19.3	1.14	5.7	20.6	1.20	6.1	21.8
100	1.15	9.1	32.6	1.24	9.7	35.0	1.32	10.4	37.3	1.40	11.0	39.5
125	1.34	16.4	59.0	1.43	17.6	63.3	1.53	18.7	67.4	1.61	19.8	71.3
150	1.50	26.6	95.7	1.61	28.5	102.7	1.72	30.4	109.3	1.82	32.1	115.6
200	1.81	56.9	204.8	1.94	61.0	219.7	2.07	64.9	233.8	2.19	68.6	247.1
250	2.09	102.5	368.9	2.24	109.9	395.7	2.38	116.9	421.0	2.52	123.6	444.9
300	2.34	165.6	596.2	2.51	177.6	639.4	2.67	188.9	680.0	2.82	199.6	718.5
350	2.58	248.4	894.1	2.77	266.3	958.7	2.94	283.2	1019.5	3.11	299.2	1077.0
400	2.81	352.7	1269.6	3.01	378.1	1361.1	3.20	402.0	1447.2	3.38	424.6	1528.6
450	3.02	480.3	1729.2	3.24	514.9	1853.6	3.44	547.4	1970.6	3.64	578.1	2081.3
500	3.22	633.1	2279.1	3.46	678.6	2442.8	3.67	721.3	2596.6	3.88	761.7	2742.3
600	3.61	1020.3	3673.2	3.87	1093.4	3936.2	4.11	1162.1	4183.5	4.34	1227.1	4417.4
700	3.97	1526.8	5496.5	4.25	1635.9	5889.1	4.52	1738.4	6258.2	4.77	1835.4	6607.4
800	4.31	2164.0	7790.3	4.61	2318.3	8345.8	4.90	2463.3	8867.8	5.17	2600.5	9361.8
900	4.63	2942.7	10,593.7	4.95	3152.2	11,347.8	5.26	3349.0	12,056.5	5.56	3535.3	12,727.1
1000	4.93	3873.2	13,943.6	5.28	4148.6	14,934.8	5.61	4407.3	15,866.3	5.92	4652.1	16,747.6
1100	5.22	4965.3	17,875.2	5.60	5317.9	19,144.4	5.94	5649.2	20,337.0	6.27	5962.6	21,465.4
1200	5.51	6228.4	22,422.3	5.90	6670.2	24,012.6	6.26	7085.2	25,506.9	6.61	7478.0	26,920.7
1300	5.78	7671.4	27,617.2	6.19	8215.0	29,574.1	6.57	8725.8	31,412.7	6.94	9209.0	33,152.4
1400	6.04	9303.1	33,491.3	6.47	9961.8	35,862.3	6.87	10,580.6	38,090.1	7.25	11,166.1	40,197.9
1500	6.30	11,131.9	40,074.8	6.74	11,919.4	42,909.7	7.16	12,659.2	45,573.3	7.56	13,359.3	48,093.3
1600	6.55	13,165.9	47,397.1	7.01	14,096.6	50,747.6	7.45	14,971.0	53,895.6	7.86	15,798.3	56,873.8

k(mm) = 0.1 T(°C) = 10

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.11	0.2	0.8	0.16	0.3	1.1	0.24	0.5	1.7	0.30	0.6	2.1
80	0.15	0.8	2.8	0.23	1.1	4.1	0.33	1.7	6.0	0.42	2.1	7.5
100	0.18	1.4	5.1	0.27	2.1	7.5	0.39	3.1	11.0	0.48	3.8	13.7
125	0.21	2.6	9.3	0.31	3.8	13.7	0.45	5.6	20.0	0.56	6.9	24.9
150	0.24	4.2	15.3	0.35	6.2	22.3	0.51	9.0	32.5	0.64	11.2	40.5
200	0.29	9.2	33.0	0.43	13.4	48.2	0.62	19.5	70.0	0.77	24.2	87.0
250	0.34	16.7	60.0	0.49	24.3	87.4	0.72	35.2	126.7	0.89	43.7	157.2
300	0.38	27.1	97.6	0.56	39.4	141.9	0.81	57.1	205.4	1.00	70.7	254.6
350	0.42	40.9	147.2	0.62	59.3	213.6	0.89	85.8	308.8	1.10	106.2	382.5
400	0.46	58.3	209.9	0.67	84.5	304.3	0.97	122.1	439.4	1.20	151.1	543.9
450	0.50	79.7	286.9	0.73	115.4	415.5	1.05	166.5	599.6	1.30	206.0	741.8
500	0.54	105.4	379.3	0.78	152.5	548.9	1.12	219.8	791.5	1.38	271.9	978.7
600	0.60	170.7	614.6	0.87	246.7	888.1	1.26	355.2	1278.8	1.55	439.0	1580.3
700	0.67	256.5	923.5	0.96	370.3	1333.1	1.38	532.6	1917.5	1.71	657.8	2368.2
800	0.73	364.9	1313.6	1.05	526.2	1894.3	1.50	756.2	2722.3	1.86	933.5	3360.6
900	0.78	497.7	1791.7	1.13	717.1	2581.5	1.62	1029.8	3707.2	2.00	1270.7	4574.7
1000	0.84	656.8	2364.4	1.20	945.6	3404.3	1.73	1357.1	4885.6	2.13	1674.1	6026.7
1100	0.89	843.9	3037.9	1.28	1214.3	4371.4	1.83	1741.7	6269.9	2.26	2147.8	7732.1
1200	0.94	1060.7	3818.5	1.35	1525.4	5491.4	1.93	2186.8	7872.5	2.38	2696.1	9705.9
1300	0.99	1308.8	4711.7	1.42	1881.3	6772.6	2.03	2695.8	9704.8	2.50	3322.8	11,962.2
1400	1.03	1589.8	5723.3	1.48	2284.1	8222.9	2.13	3271.7	11,778.3	2.62	4031.9	14,514.8
1500	1.08	1905.2	6858.7	1.55	2736.1	9849.9	2.22	3917.7	14,103.6	2.73	4827.0	17,377.2
1600	1.12	2256.4	8123.0	1.61	3239.2	11,661.3	2.31	4636.5	16,691.5	2.84	5711.7	20,562.2

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.35	0.7	2.5	0.40	0.8	2.8	0.44	0.9	3.1	0.48	0.9	3.4
80	0.49	2.4	8.8	0.55	2.8	9.9	0.61	3.0	11.0	0.66	3.3	11.9
100	0.57	4.4	16.0	0.64	5.0	18.0	0.70	5.5	19.9	0.76	6.0	21.6
125	0.66	8.1	29.0	0.74	9.1	32.7	0.82	10.0	36.1	0.89	10.9	39.1
150	0.74	13.1	47.2	0.84	14.8	53.2	0.92	16.3	58.6	1.00	17.6	63.5
200	0.90	28.1	101.3	1.01	31.7	114.0	1.11	34.9	125.5	1.20	37.8	136.1
250	1.04	50.8	183.0	1.16	57.2	205.8	1.28	62.9	226.4	1.39	68.2	245.5
300	1.16	82.3	296.2	1.31	92.5	333.0	1.44	101.7	366.3	1.56	110.3	397.0
350	1.28	123.6	444.8	1.44	138.8	499.9	1.59	152.7	549.7	1.72	165.4	595.6
400	1.40	175.6	632.3	1.57	197.3	710.4	1.73	217.0	781.0	1.87	235.0	846.1
450	1.51	239.5	862.0	1.69	268.9	968.2	1.86	295.6	1064.3	2.01	320.2	1152.8
500	1.61	315.9	1137.1	1.81	354.7	1276.9	1.99	389.8	1403.4	2.15	422.2	1519.9
600	1.80	509.8	1835.1	2.02	572.2	2060.0	2.22	628.8	2263.6	2.41	680.8	2450.9
700	1.98	763.6	2749.1	2.23	857.0	3085.1	2.45	941.5	3389.3	2.65	1019.2	3669.2
800	2.16	1083.3	3899.9	2.42	1215.5	4375.7	2.66	1335.1	4806.2	2.87	1445.1	5202.4
900	2.32	1474.3	5307.5	2.60	1653.8	5953.9	2.86	1816.3	6538.7	3.09	1965.8	7076.9
1000	2.47	1941.8	6990.5	2.77	2178.0	7840.6	3.05	2391.6	8609.8	3.30	2588.2	9317.5
1100	2.62	2490.8	8967.0	2.94	2793.3	10,056.1	3.23	3067.0	11,041.3	3.49	3318.9	11,947.9
1200	2.76	3126.1	11,254.0	3.10	3505.4	12,619.3	3.40	3848.5	13,854.4	3.68	4164.1	14,990.8
1300	2.90	3852.3	13,868.1	3.25	4319.1	15,548.8	3.57	4741.5	17,069.3	3.86	5130.0	18,468.0
1400	3.04	4673.7	16,825.2	3.40	5239.6	18,862.5	3.74	5751.5	20,705.3	4.04	6222.4	22,400.7
1500	3.17	5594.6	20,140.7	3.55	6271.5	22,577.4	3.90	6883.8	24,781.6	4.21	7447.0	26,809.3
1600	3.29	6619.3	23,829.6	3.69	7419.6	26,710.4	4.05	8143.4	29,316.2	4.38	8809.3	31,713.4

k(mm) = 0.1 T(°C) = 10

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.51	1.0	3.6	0.55	1.1	3.9	0.58	1.1	4.1	0.64	1.3	4.5
80	0.71	3.6	12.8	0.75	3.8	13.6	0.80	4.0	14.4	0.88	4.4	15.9
100	0.82	6.4	23.2	0.87	6.9	24.7	0.92	7.3	26.1	1.02	8.0	28.8
125	0.95	11.7	42.0	1.01	12.4	44.7	1.07	13.1	47.3	1.18	14.5	52.0
150	1.07	18.9	68.2	1.14	20.1	72.5	1.21	21.3	76.7	1.33	23.4	84.4
200	1.29	40.6	146.0	1.37	43.1	155.3	1.45	45.6	164.1	1.60	50.1	180.5
250	1.49	73.1	263.2	1.58	77.7	279.9	1.67	82.1	295.7	1.84	90.3	325.0
300	1.67	118.2	425.5	1.78	125.7	452.4	1.88	132.7	477.8	2.06	145.9	525.2
350	1.84	177.3	638.4	1.96	188.5	678.6	2.07	199.1	716.6	2.27	218.7	787.5
400	2.00	251.9	906.7	2.13	267.7	963.7	2.25	282.7	1017.6	2.47	310.5	1118.0
450	2.16	343.1	1235.3	2.29	364.6	1312.7	2.42	385.0	1386.0	2.66	422.9	1522.5
500	2.30	452.3	1628.4	2.45	480.7	1730.4	2.58	507.5	1826.9	2.84	557.3	2006.4
600	2.58	729.3	2625.4	2.74	774.8	2789.4	2.89	817.9	2944.6	3.18	898.1	3233.3
700	2.84	1091.6	3929.8	3.01	1159.7	4174.8	3.18	1224.0	4406.5	3.49	1343.8	4837.7
800	3.08	1547.6	5571.3	3.27	1643.9	5918.0	3.45	1735.0	6246.0	3.79	1904.5	6856.2
900	3.31	2105.0	7578.0	3.51	2235.8	8048.9	3.71	2359.5	8494.4	4.07	2589.8	9323.2
1000	3.53	2771.2	9976.5	3.75	2943.2	10,595.6	3.95	3105.9	11,181.3	4.34	3408.6	12,271.1
1100	3.74	3553.3	12,792.0	3.97	3773.6	13,585.0	4.19	3982.0	14,335.3	4.60	4369.7	15,731.1
1200	3.94	4458.0	16,048.8	4.19	4734.1	17,042.9	4.42	4995.4	17,983.3	4.85	5481.3	19,732.8
1300	4.14	5491.8	19,770.4	4.39	5831.7	20,994.0	4.64	6153.2	22,151.5	5.09	6751.4	24,304.9
1400	4.33	6660.9	23,979.3	4.59	7072.8	25,462.2	4.85	7462.5	26,865.1	5.32	8187.5	29,475.0
1500	4.51	7971.4	28,697.2	4.79	8464.1	30,470.7	5.05	8930.1	32,148.5	5.54	9797.2	35,269.8
1600	4.69	9429.2	33,945.3	4.98	10,011.6	36,041.9	5.25	10,562.6	38,025.4	5.76	11,587.5	41,715.2

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.69	1.4	4.9	0.75	1.5	5.3	0.79	1.6	5.6	0.84	1.6	5.9
80	0.95	4.8	17.2	1.02	5.1	18.5	1.09	5.5	19.7	1.15	5.8	20.8
100	1.10	8.7	31.2	1.18	9.3	33.5	1.26	9.9	35.6	1.33	10.5	37.7
125	1.28	15.7	56.4	1.37	16.8	60.5	1.46	17.9	64.4	1.54	18.9	68.0
150	1.44	25.4	91.5	1.54	27.2	98.1	1.64	29.0	104.3	1.73	30.6	110.2
200	1.73	54.3	195.6	1.85	58.2	209.6	1.97	61.9	222.8	2.08	65.4	235.3
250	1.99	97.8	352.1	2.14	104.8	377.3	2.27	111.4	401.0	2.40	117.6	423.4
300	2.24	158.0	568.8	2.39	169.3	609.3	2.54	179.9	647.5	2.69	189.9	683.6
350	2.46	236.8	852.6	2.64	253.7	913.3	2.80	269.5	970.4	2.96	284.5	1024.3
400	2.68	336.2	1210.3	2.87	360.1	1296.3	3.04	382.5	1377.1	3.21	403.8	1453.6
450	2.88	457.8	1648.0	3.08	490.3	1764.9	3.27	520.8	1874.8	3.46	549.6	1978.7
500	3.07	603.2	2171.7	3.29	646.0	2325.5	3.49	686.1	2470.1	3.69	724.1	2606.8
600	3.44	971.9	3498.9	3.68	1040.6	3746.2	3.91	1105.2	3978.6	4.12	1166.2	4198.5
700	3.78	1454.0	5234.4	4.04	1556.6	5603.8	4.30	1653.0	5950.8	4.53	1744.2	6279.1
800	4.10	2060.5	7417.6	4.39	2205.7	7940.3	4.66	2342.1	8431.4	4.92	2471.1	8895.9
900	4.40	2801.6	10,085.7	4.71	2998.8	10,795.5	5.00	3184.0	11,462.4	5.28	3359.2	12,093.2
1000	4.69	3687.1	13,273.6	5.02	3946.4	14,206.9	5.33	4189.9	15,083.7	5.63	4420.3	15,913.1
1100	4.97	4726.4	17,015.1	5.32	5058.5	18,210.4	5.65	5370.4	19,333.3	5.96	5665.4	20,395.6
1200	5.24	5928.4	21,342.1	5.61	6344.5	22,840.3	5.96	6735.5	24,247.7	6.28	7105.3	25,579.0
1300	5.50	7301.6	26,285.7	5.89	7813.8	28,129.7	6.25	8295.0	29,861.9	6.59	8750.1	31,500.5
1400	5.75	8854.3	31,875.6	6.16	9475.1	34,110.5	6.53	10,058.3	36,209.8	6.89	10,609.9	38,195.5
1500	6.00	10,594.6	38,140.7	6.42	11,337.0	40,813.4	6.81	12,034.4	43,323.9	7.18	12,694.1	45,698.7
1600	6.23	12,530.3	45,109.0	6.67	13,407.9	48,268.4	7.08	14,232.3	51,236.1	7.47	15,012.0	54,043.3

k(mm) = 0.5 T(°C) = 10

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.10	0.2	0.7	0.15	0.3	1.0	0.21	0.4	1.5	0.26	0.5	1.8
80	0.14	0.7	2.5	0.20	1.0	3.7	0.29	1.5	5.3	0.36	1.8	6.6
100	0.16	1.3	4.6	0.24	1.9	6.7	0.34	2.7	9.7	0.42	3.3	11.9
125	0.19	2.4	8.5	0.28	3.4	12.2	0.40	4.9	17.6	0.49	6.0	21.7
150	0.22	3.8	13.8	0.31	5.5	19.9	0.45	7.9	28.6	0.55	9.8	35.2
200	0.26	8.3	29.9	0.38	11.9	42.9	0.54	17.1	61.4	0.67	21.0	75.7
250	0.31	15.0	54.2	0.44	21.6	77.7	0.63	30.9	111.1	0.77	38.0	136.7
300	0.35	24.4	88.0	0.50	35.0	126.0	0.71	50.0	180.0	0.87	61.5	221.5
350	0.38	36.8	132.5	0.55	52.7	189.6	0.78	75.2	270.5	0.96	92.4	332.7
400	0.42	52.4	188.7	0.60	75.0	269.9	0.85	106.9	384.9	1.05	131.4	473.2
450	0.45	71.6	257.8	0.64	102.3	368.4	0.92	145.9	525.1	1.13	179.3	645.4
500	0.48	94.6	340.6	0.69	135.1	486.4	0.98	192.5	693.0	1.20	236.6	851.7
600	0.54	153.1	551.2	0.77	218.5	786.5	1.10	311.0	1119.7	1.35	382.1	1375.6
700	0.60	229.9	827.5	0.85	327.8	1179.9	1.21	466.4	1679.0	1.49	572.8	2062.1
800	0.65	326.7	1176.0	0.93	465.6	1676.0	1.32	662.2	2383.8	1.62	813.1	2927.2
900	0.70	445.3	1603.0	1.00	634.3	2283.5	1.42	901.9	3246.7	1.74	1107.2	3986.1
1000	0.75	587.3	2114.3	1.06	836.3	3010.6	1.51	1188.7	4279.2	1.86	1459.2	5253.0
1100	0.79	754.3	2715.5	1.13	1073.7	3865.2	1.61	1525.7	5492.5	1.97	1872.7	6741.6
1200	0.84	947.7	3411.8	1.19	1348.6	4855.0	1.69	1915.9	6897.4	2.08	2351.4	8465.1
1300	0.88	1169.1	4208.6	1.25	1663.1	5987.2	1.78	2362.3	8504.2	2.18	2898.9	10,436.1
1400	0.92	1419.6	5110.7	1.31	2019.1	7268.8	1.86	2867.4	10,322.7	2.29	3518.5	12,666.8
1500	0.96	1700.8	6122.9	1.37	2418.5	8706.7	1.94	3434.1	12,362.8	2.38	4213.6	15,168.9
1600	1.00	2013.9	7250.0	1.42	2863.2	10,307.4	2.02	4064.9	14,633.6	2.48	4987.2	17,954.1

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.30	0.6	2.1	0.34	0.7	2.4	0.38	0.7	2.7	0.41	0.8	2.9
80	0.42	2.1	7.6	0.47	2.4	8.6	0.52	2.6	9.4	0.56	2.8	10.2
100	0.49	3.8	13.9	0.55	4.3	15.5	0.60	4.7	17.1	0.65	5.1	18.5
125	0.57	7.0	25.1	0.64	7.8	28.2	0.70	8.6	31.0	0.76	9.3	33.5
150	0.64	11.3	40.8	0.72	12.7	45.8	0.79	14.0	50.3	0.86	15.1	54.4
200	0.78	24.4	87.7	0.87	27.3	98.3	0.95	30.0	107.9	1.03	32.4	116.7
250	0.90	44.0	158.4	1.00	49.3	177.4	1.10	54.1	194.7	1.19	58.5	210.5
300	1.01	71.2	256.4	1.13	79.8	287.2	1.24	87.5	315.1	1.34	94.6	340.7
350	1.11	107.0	385.2	1.25	119.8	431.4	1.37	131.4	473.2	1.48	142.1	511.6
400	1.21	152.1	547.7	1.36	170.4	613.3	1.49	186.9	672.7	1.61	202.0	727.3
450	1.30	207.5	746.9	1.46	232.3	836.3	1.60	254.8	917.2	1.73	275.4	991.5
500	1.39	273.7	985.5	1.56	306.5	1103.4	1.71	336.1	1210.0	1.85	363.3	1308.0
600	1.56	442.0	1591.4	1.75	494.9	1781.5	1.92	542.6	1953.4	2.07	586.5	2111.5
700	1.72	662.6	2385.2	1.93	741.6	2669.9	2.11	813.1	2927.3	2.28	878.9	3164.0
800	1.87	940.4	3385.4	2.09	1052.5	3789.2	2.30	1153.9	4154.2	2.48	1247.2	4489.9
900	2.01	1280.4	4609.5	2.25	1433.0	5158.9	2.47	1571.0	5655.6	2.67	1697.9	6112.3
1000	2.15	1687.2	6074.1	2.40	1888.2	6797.5	2.64	2069.9	7451.6	2.85	2237.0	8053.2
1100	2.28	2165.2	7794.8	2.55	2423.0	8722.8	2.79	2656.1	9561.8	3.02	2870.4	10,333.4
1200	2.40	2718.6	9786.9	2.69	3042.1	10,951.6	2.95	3334.6	12,004.6	3.19	3603.6	12,972.9
1300	2.52	3351.4	12,065.0	2.83	3750.1	13,500.2	3.10	4110.5	14,797.8	3.35	4442.0	15,991.1
1400	2.64	4067.5	14,643.1	2.96	4551.2	16,384.4	3.24	4988.5	17,958.8	3.50	5390.7	19,406.6
1500	2.76	4870.8	17,534.9	3.08	5449.9	19,619.5	3.38	5973.4	21,504.2	3.65	6454.8	23,237.4
1600	2.87	5764.9	20,753.6	3.21	6450.1	23,220.2	3.52	7069.5	25,450.2	3.80	7639.2	27,501.0

k(mm) = 0.5 T(°C) = 10

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.44	0.9	3.1	0.46	0.9	3.3	0.49	1.0	3.5	0.54	1.1	3.8
80	0.60	3.0	10.9	0.64	3.2	11.6	0.68	3.4	12.2	0.74	3.7	13.4
100	0.70	5.5	19.8	0.74	5.8	21.0	0.78	6.2	22.2	0.86	6.8	24.4
125	0.81	10.0	35.9	0.86	10.6	38.1	0.91	11.2	40.2	1.00	12.3	44.1
150	0.92	16.2	58.2	0.97	17.2	61.8	1.03	18.1	65.2	1.13	19.9	71.6
200	1.10	34.7	124.9	1.17	36.8	132.6	1.24	38.8	139.8	1.36	42.6	153.4
250	1.27	62.6	225.3	1.35	66.4	239.2	1.43	70.1	252.3	1.57	76.9	276.7
300	1.43	101.3	364.6	1.52	107.5	387.0	1.60	113.4	408.2	1.76	124.3	447.6
350	1.58	152.1	547.4	1.68	161.4	581.0	1.77	170.2	612.8	1.94	186.7	672.0
400	1.72	216.1	778.1	1.83	229.4	825.8	1.93	241.9	871.0	2.11	265.3	955.0
450	1.85	294.7	1060.8	1.97	312.7	1125.8	2.07	329.8	1187.3	2.27	361.6	1301.7
500	1.98	388.7	1399.3	2.10	412.5	1485.0	2.22	435.0	1566.1	2.43	476.9	1716.9
600	2.22	627.4	2258.7	2.35	665.8	2396.9	2.48	702.1	2527.6	2.72	769.7	2770.8
700	2.44	940.1	3384.3	2.59	997.6	3591.2	2.73	1051.9	3787.0	3.00	1153.1	4151.1
800	2.65	1334.0	4802.3	2.82	1415.5	5095.8	2.97	1492.6	5373.4	3.25	1636.0	5889.7
900	2.85	1816.0	6537.5	3.03	1926.9	6936.8	3.19	2031.8	7314.5	3.50	2226.9	8017.0
1000	3.05	2392.5	8613.1	3.23	2538.6	9139.0	3.41	2676.8	9636.4	3.74	2933.8	10,561.6
1100	3.23	3069.9	11,051.5	3.43	3257.2	11,726.1	3.61	3434.5	12,364.1	3.96	3764.1	13,550.7
1200	3.41	3853.9	13,874.2	3.62	4089.1	14,720.8	3.81	4311.5	15,521.5	4.18	4725.2	17,010.8
1300	3.58	4750.5	17,101.8	3.80	5040.3	18,145.0	4.00	5314.4	19,131.7	4.39	5824.2	20,967.0
1400	3.75	5765.0	20,754.2	3.97	6116.6	22,019.9	4.19	6449.2	23,217.0	4.59	7067.7	25,443.7
1500	3.91	6903.0	24,850.6	4.14	7323.8	26,365.8	4.37	7721.9	27,799.0	4.79	8462.4	30,464.6
1600	4.06	8169.4	29,409.9	4.31	8667.4	31,202.8	4.55	9138.5	32,898.5	4.98	10,014.6	36,052.5

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.58	1.1	4.1	0.62	1.2	4.4	0.66	1.3	4.7	0.70	1.4	5.0
80	0.80	4.0	14.5	0.86	4.3	15.6	0.91	4.6	16.5	0.96	4.8	17.4
100	0.93	7.3	26.3	1.00	7.8	28.2	1.06	8.3	29.9	1.12	8.8	31.6
125	1.08	13.3	47.7	1.16	14.2	51.1	1.23	15.1	54.2	1.29	15.9	57.2
150	1.22	21.5	77.4	1.30	23.0	82.8	1.38	24.4	87.9	1.46	25.8	92.8
200	1.47	46.1	165.9	1.57	49.3	177.5	1.67	52.3	188.4	1.76	55.2	198.7
250	1.69	83.1	299.1	1.81	88.9	320.0	1.92	94.3	339.6	2.03	99.5	358.2
300	1.90	134.4	483.9	2.03	143.8	517.7	2.16	152.6	549.4	2.28	160.9	579.3
350	2.10	201.8	726.3	2.24	215.8	777.0	2.38	229.0	824.5	2.51	241.5	869.5
400	2.28	286.7	1032.2	2.44	306.7	1104.1	2.59	325.4	1171.6	2.73	343.2	1235.4
450	2.46	390.8	1406.9	2.63	418.0	1504.8	2.79	443.5	1596.8	2.94	467.7	1683.8
500	2.63	515.4	1855.6	2.81	551.3	1984.7	2.98	585.0	2105.9	3.14	616.8	2220.6
600	2.94	831.8	2994.4	3.15	889.6	3202.6	3.34	943.9	3398.1	3.52	995.3	3583.0
700	3.24	1246.1	4485.9	3.46	1332.6	4797.5	3.67	1414.0	5090.2	3.87	1490.9	5367.1
800	3.52	1767.9	6364.5	3.76	1890.7	6806.5	3.99	2006.0	7221.6	4.21	2115.1	7614.2
900	3.78	2406.4	8663.0	4.05	2573.4	9264.4	4.29	2730.3	9829.2	4.53	2878.7	10,363.4
1000	4.04	3170.1	11,412.3	4.32	3390.1	12,204.3	4.58	3596.7	12,948.1	4.83	3792.1	13,651.6
1100	4.28	4067.2	14,642.0	4.58	4349.4	15,657.7	4.86	4614.4	16,611.8	5.12	4865.0	17,514.1
1200	4.51	5105.7	18,380.3	4.83	5459.8	19,655.1	5.12	5792.3	20,852.4	5.40	6106.9	21,984.9
1300	4.74	6293.0	22,654.7	5.07	6729.3	24,225.6	5.38	7139.2	25,701.0	5.67	7526.8	27,096.5
1400	4.96	7636.5	27,491.3	5.30	8165.9	29,397.3	5.63	8663.2	31,187.4	5.93	9133.5	32,880.5
1500	5.17	9143.3	32,915.9	5.53	9777.1	35,197.5	5.87	10,372.4	37,340.5	6.19	10,935.4	39,367.4
1600	5.38	10,820.3	38,953.0	5.75	11,570.2	41,652.7	6.10	12,274.5	44,188.3	6.44	12,940.7	46,586.6

k(mm) = 1 T(°C) = 10

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.09	0.2	0.7	0.13	0.3	0.9	0.19	0.4	1.4	0.24	0.5	1.7
80	0.13	0.7	2.3	0.19	0.9	3.4	0.27	1.3	4.8	0.33	1.7	6.0
100	0.15	1.2	4.3	0.22	1.7	6.2	0.31	2.5	8.8	0.38	3.0	10.9
125	0.18	2.2	7.8	0.25	3.1	11.2	0.36	4.5	16.1	0.45	5.5	19.8
150	0.20	3.6	12.8	0.29	5.1	18.3	0.41	7.3	26.2	0.51	8.9	32.2
200	0.24	7.7	27.7	0.35	11.0	39.6	0.50	15.7	56.4	0.61	19.3	69.3
250	0.28	14.0	50.3	0.41	19.9	71.7	0.58	28.4	102.1	0.71	34.8	125.4
300	0.32	22.7	81.7	0.46	32.3	116.5	0.65	46.0	165.7	0.80	56.5	203.5
350	0.36	34.2	123.1	0.51	48.7	175.3	0.72	69.2	249.3	0.88	85.0	306.0
400	0.39	48.7	175.4	0.55	69.4	249.7	0.78	98.6	354.9	0.96	121.0	435.7
450	0.42	66.6	239.6	0.60	94.7	341.1	0.85	134.6	484.6	1.04	165.2	594.7
500	0.45	88.0	316.7	0.64	125.2	450.6	0.91	177.8	640.0	1.11	218.2	785.4
600	0.50	142.5	512.8	0.72	202.6	729.2	1.02	287.6	1035.3	1.25	352.8	1270.2
700	0.56	214.0	770.4	0.79	304.1	1094.8	1.12	431.6	1553.9	1.38	529.5	1906.2
800	0.61	304.3	1095.4	0.86	432.3	1556.2	1.22	613.4	2208.1	1.50	752.3	2708.5
900	0.65	414.9	1493.8	0.93	589.3	2121.5	1.31	836.0	3009.6	1.61	1025.3	3691.1
1000	0.70	547.5	1971.0	0.99	777.4	2798.6	1.40	1102.6	3969.3	1.72	1352.2	4867.8
1100	0.74	703.4	2532.3	1.05	998.6	3594.8	1.49	1416.1	5097.8	1.83	1736.5	6251.3
1200	0.78	884.1	3182.7	1.11	1254.8	4517.3	1.57	1779.2	6405.2	1.93	2181.6	7853.9
1300	0.82	1090.9	3927.2	1.17	1548.1	5573.1	1.65	2194.8	7901.1	2.03	2691.0	9687.7
1400	0.86	1325.1	4770.4	1.22	1880.2	6768.6	1.73	2665.3	9595.1	2.12	3267.8	11,764.1
1500	0.90	1588.0	5716.8	1.27	2252.9	8110.4	1.81	3193.4	11,496.1	2.22	3915.1	14,094.2
1600	0.94	1880.8	6770.8	1.33	2668.0	9604.7	1.88	3781.4	13,613.0	2.31	4635.8	16,688.9

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.27	0.5	1.9	0.31	0.6	2.2	0.34	0.7	2.4	0.37	0.7	2.6
80	0.38	1.9	6.9	0.43	2.2	7.7	0.47	2.4	8.5	0.51	2.6	9.2
100	0.45	3.5	12.6	0.50	3.9	14.1	0.55	4.3	15.5	0.59	4.7	16.7
125	0.52	6.4	22.9	0.58	7.1	25.6	0.64	7.8	28.1	0.69	8.5	30.4
150	0.59	10.4	37.3	0.66	11.6	41.7	0.72	12.7	45.8	0.78	13.7	49.5
200	0.71	22.3	80.2	0.79	24.9	89.8	0.87	27.4	98.5	0.94	29.6	106.5
250	0.82	40.3	145.1	0.92	45.1	162.4	1.01	49.5	178.1	1.09	53.5	192.5
300	0.92	65.4	235.3	1.04	73.2	263.4	1.13	80.2	288.8	1.23	86.7	312.1
350	1.02	98.3	353.9	1.14	110.0	396.1	1.25	120.6	434.2	1.35	130.4	469.3
400	1.11	139.9	503.8	1.25	156.6	563.8	1.37	171.7	618.0	1.48	185.5	667.9
450	1.20	191.0	687.6	1.34	213.7	769.5	1.47	234.3	843.4	1.59	253.2	911.5
500	1.28	252.2	908.0	1.44	282.2	1016.0	1.58	309.4	1113.7	1.70	334.3	1203.5
600	1.44	407.9	1468.3	1.61	456.3	1642.8	1.77	500.2	1800.6	1.91	540.5	1945.7
700	1.59	612.0	2203.3	1.78	684.7	2465.0	1.95	750.4	2701.6	2.11	810.9	2919.2
800	1.73	869.5	3130.3	1.94	972.8	3501.9	2.12	1066.1	3837.9	2.29	1151.9	4146.9
900	1.86	1184.9	4265.7	2.08	1325.5	4772.0	2.28	1452.7	5229.7	2.47	1569.6	5650.6
1000	1.99	1562.6	5625.2	2.23	1747.9	6292.6	2.44	1915.5	6896.0	2.64	2069.7	7450.8
1100	2.11	2006.6	7223.7	2.36	2244.6	8080.4	2.59	2459.7	8855.0	2.80	2657.6	9567.3
1200	2.23	2520.9	9075.3	2.49	2819.8	10,151.4	2.73	3090.1	11,124.3	2.95	3338.6	12,018.9
1300	2.34	3109.4	11,193.9	2.62	3478.0	12,520.9	2.87	3811.3	13,720.6	3.10	4117.7	14,823.9
1400	2.45	3775.7	13,592.7	2.74	4223.3	15,203.8	3.01	4627.9	16,660.3	3.25	4999.9	17,999.8
1500	2.56	4523.5	16,284.6	2.86	5059.6	18,214.4	3.14	5544.2	19,959.1	3.39	5989.9	21,563.6
1600	2.66	5356.1	19,282.1	2.98	5990.8	21,566.8	3.26	6564.5	23,632.4	3.53	7092.2	25,531.8

k(mm) = 1 T(°C) = 10

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.39	0.8	2.8	0.42	0.8	2.9	0.44	0.9	3.1	0.48	0.9	3.4
80	0.54	2.7	9.8	0.58	2.9	10.5	0.61	3.1	11.0	0.67	3.4	12.1
100	0.63	5.0	17.9	0.67	5.3	19.0	0.71	5.6	20.1	0.78	6.1	22.0
125	0.74	9.0	32.6	0.78	9.6	34.6	0.82	10.1	36.4	0.90	11.1	40.0
150	0.83	14.7	53.0	0.88	15.6	56.2	0.93	16.5	59.3	1.02	18.1	65.0
200	1.01	31.6	113.9	1.07	33.6	120.9	1.13	35.4	127.5	1.24	38.8	139.7
250	1.17	57.2	205.9	1.24	60.7	218.5	1.30	64.0	230.4	1.43	70.2	252.6
300	1.31	92.7	333.8	1.39	98.4	354.2	1.47	103.8	373.5	1.61	113.7	409.4
350	1.45	139.4	501.9	1.54	147.9	532.6	1.62	156.0	561.6	1.78	171.0	615.5
400	1.58	198.4	714.3	1.68	210.5	757.9	1.77	222.0	799.2	1.94	243.3	875.9
450	1.70	270.8	974.8	1.81	287.3	1034.3	1.90	302.9	1090.6	2.09	332.0	1195.2
500	1.82	357.5	1287.1	1.93	379.3	1365.6	2.04	400.0	1439.9	2.23	438.3	1578.0
600	2.04	578.0	2080.8	2.17	613.2	2207.6	2.29	646.6	2327.6	2.51	708.5	2550.7
700	2.25	867.1	3121.7	2.39	920.0	3311.9	2.52	970.0	3491.8	2.76	1062.9	3826.5
800	2.45	1231.8	4434.5	2.60	1306.8	4704.6	2.74	1377.8	4960.1	3.00	1509.8	5435.3
900	2.64	1678.4	6042.3	2.80	1780.6	6410.3	2.95	1877.3	6758.3	3.23	2057.1	7405.6
1000	2.82	2213.1	7967.2	2.99	2347.9	8452.3	3.15	2475.3	8911.1	3.45	2712.3	9764.4
1100	2.99	2841.7	10,230.3	3.17	3014.7	10,853.0	3.34	3178.3	11,442.0	3.66	3482.6	12,537.4
1200	3.16	3569.9	12,851.7	3.35	3787.2	13,633.8	3.53	3992.7	14,373.5	3.87	4374.8	15,749.4
1300	3.32	4403.0	15,850.8	3.52	4670.9	16,815.3	3.71	4924.3	17,727.5	4.07	5395.6	19,424.3
1400	3.47	5346.3	19,246.5	3.68	5671.5	20,417.5	3.88	5979.2	21,525.0	4.26	6551.4	23,584.9
1500	3.62	6404.7	23,057.0	3.84	6794.3	24,459.6	4.05	7162.8	25,786.2	4.44	7848.2	28,253.7
1600	3.77	7583.3	27,299.8	4.00	8044.5	28,960.4	4.22	8480.8	30,531.0	4.62	9292.3	33,452.1

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.52	1.0	3.7	0.56	1.1	3.9	0.59	1.2	4.2	0.63	1.2	4.4
80	0.72	3.6	13.1	0.77	3.9	14.0	0.82	4.1	14.9	0.87	4.4	15.7
100	0.84	6.6	23.8	0.90	7.1	25.5	0.96	7.5	27.0	1.01	7.9	28.5
125	0.98	12.0	43.2	1.05	12.8	46.2	1.11	13.6	49.0	1.17	14.4	51.7
150	1.10	19.5	70.3	1.18	20.9	75.1	1.25	22.2	79.7	1.32	23.4	84.1
200	1.34	41.9	151.0	1.43	44.9	161.5	1.52	47.6	171.4	1.60	50.2	180.7
250	1.54	75.8	273.0	1.65	81.1	292.0	1.75	86.1	309.8	1.85	90.7	326.6
300	1.74	122.9	442.4	1.86	131.4	473.2	1.97	139.5	502.0	2.08	147.0	529.3
350	1.92	184.8	665.1	2.05	197.6	711.3	2.18	209.6	754.7	2.30	221.0	795.7
400	2.09	262.9	946.4	2.24	281.1	1012.1	2.37	298.3	1073.8	2.50	314.5	1132.1
450	2.26	358.7	1291.4	2.41	383.6	1381.0	2.56	407.0	1465.1	2.70	429.1	1544.7
500	2.41	473.6	1705.0	2.58	506.4	1823.2	2.74	537.3	1934.2	2.88	566.5	2039.3
600	2.71	765.5	2755.9	2.90	818.6	2947.0	3.07	868.4	3126.4	3.24	915.6	3296.0
700	2.98	1148.4	4134.2	3.19	1227.9	4420.6	3.38	1302.7	4689.6	3.57	1373.4	4944.1
800	3.25	1631.2	5872.3	3.47	1744.2	6279.0	3.68	1850.3	6661.0	3.88	1950.6	7022.3
900	3.49	2222.5	8000.8	3.74	2376.4	8554.9	3.96	2520.9	9075.3	4.18	2657.6	9567.4
1000	3.73	2930.3	10,549.1	3.99	3133.2	11,279.4	4.23	3323.7	11,965.4	4.46	3504.0	12,614.2
1100	3.96	3762.4	13,544.8	4.23	4022.9	14,482.4	4.49	4267.5	15,363.1	4.73	4498.9	16,196.0
1200	4.18	4726.3	17,014.7	4.47	5053.4	18,192.4	4.74	5360.7	19,298.5	5.00	5651.3	20,344.7
1300	4.39	5829.0	20,984.6	4.70	6232.5	22,436.9	4.98	6611.4	23,800.9	5.25	6969.7	25,091.0
1400	4.60	7077.6	25 479.2	4.92	7567.3	27,242.4	5.21	8027.3	28,898.4	5.50	8462.4	30,464.7
1500	4.80	8478.5	30,522.7	5.13	9065.2	32,634.7	5.44	9616.2	34,618.4	5.74	10,137.4	36,494.5
1600	4.99	10,038.5	36,138.5	5.34	10,733.0	38,638.8	5.66	11,385.3	40,987.2	5.97	12,002.3	43,208.4

k(mm) = 5 T(°C) = 10

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.07	0.1	0.5	0.10	0.2	0.7	0.14	0.3	1.0	0.17	0.3	1.2
80	0.10	0.5	1.8	0.14	0.7	2.5	0.20	1.0	3.6	0.24	1.2	4.4
100	0.12	0.9	3.3	0.16	1.3	4.6	0.23	1.8	6.6	0.28	2.2	8.1
125	0.14	1.7	6.0	0.19	2.4	8.5	0.27	3.4	12.1	0.34	4.1	14.8
150	0.15	2.7	9.9	0.22	3.9	14.0	0.31	5.5	19.8	0.38	6.8	24.3
200	0.19	6.0	21.5	0.27	8.5	30.5	0.38	12.0	43.3	0.47	14.7	53.1
250	0.22	10.9	39.3	0.32	15.5	55.8	0.45	22.0	79.0	0.55	26.9	96.9
300	0.25	17.9	64.3	0.36	25.3	91.1	0.51	35.9	129.1	0.62	44.0	158.2
350	0.28	27.0	97.3	0.40	38.3	137.9	0.56	54.2	195.3	0.69	66.5	239.3
400	0.31	38.7	139.2	0.44	54.8	197.2	0.62	77.6	279.3	0.76	95.1	342.3
450	0.33	53.0	190.8	0.47	75.1	270.3	0.67	106.3	382.8	0.82	130.3	469.1
500	0.36	70.3	252.9	0.51	99.5	358.3	0.72	140.9	507.3	0.88	172.7	621.7
600	0.40	114.3	411.5	0.57	161.9	582.9	0.81	229.2	825.2	0.99	280.9	1011.1
700	0.45	172.4	620.6	0.63	244.1	878.9	0.90	345.6	1244.1	1.10	423.4	1524.4
800	0.49	246.0	885.4	0.69	348.3	1253.8	0.98	493.0	1774.6	1.20	604.0	2174.3
900	0.53	336.4	1210.9	0.75	476.2	1714.5	1.06	674.1	2426.6	1.30	825.8	2973.0
1000	0.57	444.9	1601.8	0.80	629.9	2267.7	1.14	891.5	3209.4	1.39	1092.2	3932.1
1100	0.60	572.9	2062.5	0.85	811.0	2919.8	1.21	1147.8	4132.1	1.48	1406.2	5062.4
1200	0.64	721.5	2597.5	0.90	1021.4	3676.9	1.28	1445.4	5203.4	1.57	1770.8	6374.8
1300	0.67	891.9	3210.8	0.95	1262.5	4544.9	1.35	1786.5	6431.6	1.65	2188.7	7879.3
1400	0.70	1085.2	3906.6	1.00	1536.0	5529.5	1.41	2173.5	7824.7	1.73	2662.7	9585.9
1500	0.74	1302.4	4688.7	1.04	1843.4	6636.3	1.48	2608.5	9390.7	1.81	3195.6	11,504.2
1600	0.77	1544.7	5561.0	1.09	2186.3	7870.7	1.54	3093.6	11,137.1	1.88	3789.9	13,643.5

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.19	0.4	1.4	0.22	0.4	1.5	0.24	0.5	1.7	0.26	0.5	1.8
80	0.28	1.4	5.0	0.31	1.6	5.6	0.34	1.7	6.2	0.37	1.9	6.7
100	0.33	2.6	9.3	0.37	2.9	10.4	0.40	3.2	11.4	0.44	3.4	12.3
125	0.39	4.8	17.1	0.43	5.3	19.2	0.48	5.8	21.0	0.51	6.3	22.7
150	0.44	7.8	28.1	0.49	8.7	31.5	0.54	9.6	34.5	0.59	10.3	37.2
200	0.54	17.0	61.3	0.61	19.0	68.6	0.66	20.9	75.1	0.72	22.5	81.2
250	0.63	31.1	111.9	0.71	34.8	125.2	0.78	38.1	137.2	0.84	41.2	148.2
300	0.72	50.8	182.8	0.80	56.8	204.4	0.88	62.2	224.0	0.95	67.2	242.0
350	0.80	76.8	276.5	0.89	85.9	309.2	0.98	94.1	338.8	1.06	101.7	366.0
400	0.87	109.8	395.4	0.98	122.8	442.2	1.07	134.6	484.5	1.16	145.4	523.4
450	0.95	150.5	541.9	1.06	168.3	606.0	1.16	184.4	663.9	1.25	199.2	717.2
500	1.02	199.5	718.1	1.14	223.1	803.0	1.24	244.4	879.8	1.34	264.0	950.4
600	1.15	324.4	1167.9	1.28	362.8	1306.0	1.41	397.5	1430.8	1.52	429.3	1545.6
700	1.27	489.1	1760.7	1.42	546.9	1968.8	1.56	599.2	2157.0	1.68	647.2	2330.1
800	1.39	697.6	2511.3	1.55	780.0	2808.2	1.70	854.6	3076.5	1.84	923.2	3323.3
900	1.50	953.8	3433.7	1.68	1066.5	3839.6	1.84	1168.5	4206.5	1.98	1262.2	4543.9
1000	1.61	1261.5	4541.3	1.80	1410.6	5078.0	1.97	1545.3	5563.2	2.13	1669.3	6009.4
1100	1.71	1624.1	5846.7	1.91	1816.0	6537.6	2.09	1989.5	7162.3	2.26	2149.1	7736.8
1200	1.81	2045.1	7362.3	2.02	2286.7	8232.3	2.22	2505.2	9018.8	2.39	2706.1	9742.1
1300	1.90	2527.7	9099.8	2.13	2826.4	10,175.0	2.33	3096.4	11,147.1	2.52	3344.7	12,041.1
1400	2.00	3075.2	11,070.6	2.23	3438.5	12,378.7	2.45	3767.0	13,561.3	2.64	4069.1	14,648.8
1500	2.09	3690.5	13,286.0	2.34	4126.6	14,855.7	2.56	4520.8	16,274.9	2.76	4883.3	17,580.0
1600	2.18	4376.8	15,756.5	2.43	4893.9	17,618.1	2.67	5361.4	19,301.1	2.88	5791.3	20,848.8

k(mm) = 5 T(°C) = 10

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.28	0.5	2.0	0.29	0.6	2.1	0.31	0.6	2.2	0.34	0.7	2.4
80	0.40	2.0	7.2	0.42	2.1	7.6	0.44	2.2	8.0	0.48	2.4	8.8
100	0.47	3.7	13.2	0.49	3.9	14.0	0.52	4.1	14.8	0.57	4.5	16.2
125	0.55	6.7	24.3	0.58	7.1	25.7	0.61	7.5	27.1	0.67	8.3	29.7
150	0.63	11.1	39.8	0.66	11.7	42.2	0.70	12.4	44.5	0.77	13.6	48.8
200	0.77	24.1	86.8	0.81	25.6	92.1	0.86	27.0	97.1	0.94	29.5	106.3
250	0.90	44.0	158.4	0.95	46.7	168.1	1.00	49.2	177.2	1.10	53.9	194.1
300	1.02	71.9	258.7	1.08	76.2	274.4	1.14	80.4	289.3	1.25	88.1	317.0
350	1.13	108.7	391.3	1.20	115.3	415.1	1.26	121.5	437.6	1.38	133.2	479.4
400	1.24	155.4	559.6	1.31	164.9	593.6	1.38	173.8	625.7	1.52	190.4	685.6
450	1.34	213.0	766.8	1.42	225.9	813.4	1.50	238.2	857.5	1.64	261.0	939.4
500	1.44	282.2	1016.1	1.52	299.4	1077.8	1.61	315.6	1136.2	1.76	345.8	1244.8
600	1.62	459.0	1652.5	1.72	486.9	1752.9	1.82	513.3	1847.8	1.99	562.3	2024.4
700	1.80	692.0	2491.2	1.91	734.0	2642.5	2.01	773.8	2785.6	2.20	847.7	3051.7
800	1.96	987.0	3553.1	2.08	1046.9	3768.8	2.20	1103.6	3972.9	2.41	1209.0	4352.5
900	2.12	1349.4	4858.0	2.25	1431.4	5153.0	2.37	1508.9	5432.0	2.60	1653.0	5950.9
1000	2.27	1784.7	6424.8	2.41	1893.0	6814.9	2.54	1995.5	7183.8	2.78	2186.1	7870.0
1100	2.42	2297.6	8271.4	2.56	2437.1	8773.6	2.70	2569.1	9248.6	2.96	2814.5	10,132.0
1200	2.56	2893.1	10,415.3	2.71	3068.8	11,047.7	2.86	3234.9	11,645.7	3.13	3543.9	12,758.1
1300	2.69	3575.9	12,873.2	2.86	3793.0	13,654.7	3.01	3998.3	14,393.8	3.30	4380.2	15,768.6
1400	2.83	4350.3	15,661.1	3.00	4614.4	16,611.8	3.16	4864.2	17,511.0	3.46	5328.7	19,183.4
1500	2.95	5220.8	18,794.7	3.13	5537.7	19,935.6	3.30	5837.4	21,014.7	3.62	6394.9	23,021.7
1600	3.08	6191.5	22,289.4	3.27	6567.3	23,642.4	3.44	6922.8	24,922.1	3.77	7583.9	27,302.2

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.37	0.7	2.6	0.39	0.8	2.8	0.41	0.8	2.9	0.44	0.9	3.1
80	0.52	2.6	9.5	0.56	2.8	10.1	0.59	3.0	10.8	0.63	3.1	11.3
100	0.62	4.9	17.5	0.66	5.2	18.7	0.70	5.5	19.8	0.74	5.8	20.9
125	0.73	8.9	32.1	0.78	9.5	34.3	0.82	10.1	36.4	0.87	10.7	38.4
150	0.83	14.6	52.7	0.89	15.7	56.4	0.94	16.6	59.8	0.99	17.5	63.0
200	1.02	31.9	114.9	1.09	34.1	122.8	1.15	36.2	130.3	1.21	38.2	137.4
250	1.19	58.3	209.7	1.27	62.3	224.2	1.35	66.1	237.8	1.42	69.6	250.7
300	1.35	95.1	342.4	1.44	101.7	366.1	1.53	107.9	388.3	1.61	113.7	409.4
350	1.50	143.9	517.9	1.60	153.8	553.7	1.70	163.1	587.3	1.79	172.0	619.1
400	1.64	205.7	740.6	1.75	219.9	791.8	1.86	233.3	839.8	1.96	245.9	885.3
450	1.77	281.9	1014.8	1.89	301.4	1084.9	2.01	319.7	1150.8	2.12	337.0	1213.1
500	1.90	373.5	1344.6	2.03	399.3	1437.6	2.16	423.6	1524.9	2.27	446.5	1607.4
600	2.15	607.4	2186.7	2.30	649.4	2337.9	2.44	688.8	2479.8	2.57	726.1	2614.1
700	2.38	915.7	3296.5	2.54	979.0	3524.3	2.70	1038.4	3738.2	2.84	1094.6	3940.6
800	2.60	1306.0	4701.5	2.78	1396.2	5026.4	2.95	1481.0	5331.5	3.11	1561.1	5620.1
900	2.81	1785.6	6428.1	3.00	1909.0	6872.2	3.18	2024.8	7289.4	3.36	2134.4	7683.9
1000	3.01	2361.4	8501.1	3.21	2524.6	9088.5	3.41	2677.8	9640.1	3.59	2822.8	10,161.9
1100	3.20	3040.1	10,944.4	3.42	3250.2	11,700.6	3.63	3447.4	12,410.8	3.82	3634.0	13,082.5
1200	3.38	3828.0	13,781.0	3.62	4092.5	14,733.1	3.84	4340.9	15,627.3	4.05	4575.9	16,473.1
1300	3.56	4731.3	17,032.8	3.81	5058.2	18,209.6	4.04	5365.2	19,314.8	4.26	5655.6	20,360.1
1400	3.74	5755.9	20,721.4	4.00	6153.6	22,152.9	4.24	6527.1	23,497.4	4.47	6880.3	24,769.1
1500	3.91	6907.6	24,867.4	4.18	7384.8	26,585.3	4.43	7833.0	28,198.8	4.67	8256.9	29,724.8
1600	4.07	8191.9	29,491.0	4.36	8757.8	31,528.2	4.62	9289.3	33,441.6	4.87	9792.1	35,251.4

k(mm) = 0.01 T(°C) = 20

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.12	0.2	0.8	0.17	0.3	1.2	0.26	0.5	1.8	0.33	0.6	2.3
80	0.16	0.8	3.0	0.24	1.2	4.4	0.36	1.8	6.5	0.45	2.3	8.2
100	0.19	1.5	5.5	0.29	2.2	8.1	0.42	3.3	11.9	0.53	4.2	15.0
125	0.23	2.8	10.0	0.33	4.1	14.7	0.49	6.0	21.7	0.62	7.6	27.2
150	0.26	4.5	16.3	0.38	6.7	24.0	0.56	9.8	35.4	0.70	12.3	44.3
200	0.31	9.8	35.3	0.46	14.4	52.0	0.67	21.2	76.3	0.84	26.5	95.4
250	0.36	17.8	64.2	0.53	26.2	94.3	0.78	38.4	138.2	0.98	48.0	172.6
300	0.41	29.0	104.5	0.60	42.6	153.3	0.88	62.3	224.3	1.10	77.8	280.0
350	0.46	43.8	157.6	0.67	64.2	231.0	0.97	93.8	337.7	1.22	117.0	421.3
400	0.50	62.5	224.9	0.73	91.5	329.3	1.06	133.6	481.0	1.33	166.6	599.7
450	0.54	85.5	307.7	0.79	125.0	450.0	1.15	182.5	656.9	1.43	227.4	818.7
500	0.58	113.1	407.0	0.84	165.3	594.9	1.23	241.1	867.8	1.53	300.3	1081.2
600	0.65	183.4	660.1	0.95	267.7	963.9	1.38	390.1	1404.4	1.72	485.7	1748.5
700	0.72	275.8	993.0	1.05	402.4	1448.5	1.52	585.7	2108.6	1.89	728.9	2623.9
800	0.78	392.7	1413.6	1.14	572.3	2060.4	1.66	832.5	2997.1	2.06	1035.5	3727.9
900	0.84	536.0	1929.6	1.23	780.8	2810.7	1.78	1134.9	4085.8	2.22	1411.1	5080.0
1000	0.90	707.9	2548.4	1.31	1030.5	3709.8	1.91	1497.1	5389.6	2.37	1860.7	6698.7
1100	0.96	910.2	3276.7	1.39	1324.3	4767.6	2.02	1923.0	6922.7	2.51	2389.3	8601.6
1200	1.01	1144.8	4121.4	1.47	1664.9	5993.7	2.14	2416.4	8698.9	2.65	3001.5	10,805.5
1300	1.06	1413.5	5088.7	1.55	2054.8	7397.3	2.25	2980.9	10,731.4	2.79	3701.9	13,326.9
1400	1.12	1718.0	6184.9	1.62	2496.5	8987.3	2.35	3620.2	13,032.8	2.92	4494.8	16,181.2
1500	1.17	2060.0	7416.0	1.69	2992.3	10,772.3	2.45	4337.7	15,615.6	3.05	5384.4	19,383.8
1600	1.21	2441.0	8787.7	1.76	3544.6	12,760.6	2.55	5136.6	18,491.7	3.17	6374.8	22,949.3

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.38	0.8	2.7	0.44	0.9	3.1	0.48	0.9	3.4	0.53	1.0	3.7
80	0.53	2.7	9.6	0.60	3.0	10.9	0.67	3.4	12.1	0.73	3.7	13.2
100	0.62	4.9	17.6	0.70	5.5	19.9	0.78	6.1	22.0	0.85	6.7	24.0
125	0.72	8.9	31.9	0.82	10.0	36.1	0.90	11.1	39.9	0.98	12.1	43.5
150	0.82	14.4	51.9	0.92	16.3	58.7	1.02	18.0	64.9	1.11	19.6	70.7
200	0.99	31.0	111.7	1.12	35.1	126.2	1.23	38.7	139.5	1.34	42.1	151.7
250	1.14	56.1	202.1	1.29	63.4	228.2	1.43	70.0	252.0	1.55	76.1	274.1
300	1.29	91.0	327.6	1.45	102.7	369.9	1.60	113.4	408.3	1.74	123.3	443.9
350	1.42	136.8	492.6	1.61	154.4	556.0	1.77	170.5	613.6	1.93	185.3	667.0
400	1.55	194.7	701.0	1.75	219.7	791.0	1.93	242.5	872.9	2.10	263.5	948.5
450	1.67	265.7	956.7	1.88	299.8	1079.2	2.08	330.7	1190.7	2.26	359.3	1293.7
500	1.79	350.9	1263.1	2.02	395.7	1424.6	2.22	436.5	1571.4	2.42	474.2	1707.1
600	2.01	567.2	2041.7	2.26	639.4	2302.0	2.49	705.1	2538.5	2.71	765.8	2757.0
700	2.21	850.8	3062.7	2.49	958.9	3452.0	2.75	1057.2	3805.9	2.98	1147.9	4132.6
800	2.40	1208.3	4349.9	2.71	1361.6	4901.7	2.99	1500.8	5403.0	3.24	1629.4	5865.9
900	2.59	1646.1	5926.0	2.92	1854.5	6676.2	3.21	2043.8	7357.8	3.49	2218.6	7986.9
1000	2.76	2170.1	7812.4	3.11	2444.4	8799.7	3.43	2693.5	9696.5	3.72	2923.4	10,524.2
1100	2.93	2786.0	10,029.4	3.30	3137.5	11,295.0	3.64	3456.8	12,444.4	3.95	3751.4	13,505.1
1200	3.09	3499.1	12,596.8	3.48	3940.1	14,184.2	3.84	4340.4	15,625.6	4.16	4709.9	16,955.6
1300	3.25	4314.8	15,533.3	3.66	4857.9	17,488.3	4.03	5350.9	19,263.2	4.37	5805.8	20,900.9
1400	3.40	5238.1	18,857.1	3.83	5896.6	21,227.7	4.22	6494.4	23,379.7	4.58	7045.9	25,365.2
1500	3.55	6273.8	22,585.8	4.00	7061.7	25,422.2	4.40	7776.9	27,996.8	4.77	8436.6	30,371.8
1600	3.69	7426.8	26,736.6	4.16	8358.6	30,090.9	4.58	9204.3	33,135.4	4.97	9984.3	35,943.6

k(mm) = 0.01 T(°C) = 20

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.57	1.1	4.0	0.61	1.2	4.3	0.64	1.3	4.5	0.71	1.4	5.0
80	0.78	3.9	14.2	0.84	4.2	15.2	0.89	4.5	16.1	0.98	4.9	17.8
100	0.91	7.2	25.8	0.97	7.6	27.5	1.03	8.1	29.2	1.14	9.0	32.3
125	1.06	13.0	46.8	1.13	13.9	49.9	1.20	14.7	52.9	1.32	16.2	58.4
150	1.19	21.1	76.0	1.27	22.5	81.1	1.35	23.9	85.9	1.49	26.4	94.9
200	1.44	45.3	163.2	1.54	48.3	174.0	1.63	51.2	184.3	1.80	56.5	203.4
250	1.67	81.9	294.7	1.78	87.3	314.1	1.88	92.4	332.6	2.08	102.0	367.0
300	1.88	132.6	477.2	2.00	141.3	508.6	2.12	149.5	538.3	2.33	165.0	593.9
350	2.07	199.1	716.8	2.21	212.2	763.8	2.33	224.5	808.4	2.57	247.7	891.6
400	2.25	283.1	1019.2	2.40	301.6	1085.9	2.54	319.2	1149.1	2.80	352.0	1267.2
450	2.43	386.1	1389.9	2.59	411.3	1480.6	2.74	435.2	1566.6	3.02	479.8	1727.2
500	2.59	509.4	1833.9	2.76	542.6	1953.3	2.92	574.1	2066.6	3.22	632.8	2278.1
600	2.91	822.5	2961.1	3.10	876.0	3153.4	3.28	926.6	3335.8	3.61	1021.1	3676.1
700	3.20	1232.7	4437.8	3.41	1312.6	4725.2	3.61	1388.3	4997.8	3.97	1529.5	5506.3
800	3.48	1749.5	6298.1	3.71	1862.5	6705.2	3.92	1969.8	7091.1	4.32	2169.7	7811.0
900	3.74	2381.7	8574.3	3.99	2535.4	9127.5	4.21	2681.1	9651.8	4.64	2952.7	10,629.9
1000	4.00	3138.0	11,297.0	4.25	3340.2	12,024.6	4.50	3531.7	12,714.2	4.95	3889.0	14,000.4
1100	4.24	4026.5	14,495.3	4.51	4285.4	15,427.5	4.77	4530.8	16,311.0	5.25	4988.5	17,958.6
1200	4.47	5054.8	18,197.1	4.76	5379.4	19,365.8	5.03	5687.1	20,473.5	5.54	6260.7	22,538.7
1300	4.69	6230.4	22,429.4	5.00	6630.1	23,868.2	5.28	7008.8	25,231.7	5.81	7714.9	27,773.7
1400	4.91	7560.6	27,218.0	5.23	8045.0	28,962.1	5.52	8504.1	30,614.7	6.08	9359.9	33,695.6
1500	5.12	9052.2	32,588.1	5.45	9631.7	34,674.1	5.76	10,180.7	36,650.7	6.34	11,204.2	40,335.2
1600	5.33	10,712.2	38,563.9	5.67	11,397.3	41,030.1	5.99	12,046.3	43,366.8	6.59	13,256.2	47,722.3

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.78	1.5	5.5	0.84	1.6	5.9	0.89	1.8	6.3	0.95	1.9	6.7
80	1.07	5.4	19.4	1.15	5.8	20.8	1.23	6.2	22.2	1.30	6.5	23.6
100	1.24	9.8	35.1	1.34	10.5	37.8	1.43	11.2	40.3	1.51	11.9	42.7
125	1.44	17.7	63.6	1.55	19.0	68.4	1.65	20.3	72.9	1.75	21.5	77.3
150	1.62	28.7	103.2	1.74	30.8	111.0	1.86	32.9	118.3	1.97	34.8	125.3
200	1.96	61.4	221.2	2.10	66.0	237.7	2.24	70.4	253.4	2.37	74.5	268.2
250	2.26	110.8	398.9	2.43	119.1	428.7	2.58	126.9	456.7	2.74	134.3	483.3
300	2.54	179.2	645.3	2.72	192.6	693.3	2.90	205.1	738.5	3.07	217.1	781.4
350	2.80	269.0	968.5	3.00	289.0	1040.3	3.20	307.8	1108.0	3.38	325.6	1172.2
400	3.04	382.3	1376.2	3.27	410.6	1478.0	3.48	437.2	1573.9	3.68	462.5	1664.9
450	3.28	521.0	1875.5	3.52	559.4	2014.0	3.75	595.7	2144.4	3.96	630.0	2268.1
500	3.50	687.0	2473.3	3.76	737.7	2655.6	4.00	785.4	2827.3	4.23	830.6	2990.1
600	3.92	1108.4	3990.1	4.21	1189.8	4283.3	4.48	1266.5	4559.4	4.74	1339.2	4821.0
700	4.31	1659.9	5975.5	4.63	1781.5	6413.5	4.93	1896.1	6825.8	5.21	2004.6	7216.5
800	4.68	2354.2	8475.2	5.03	2526.4	9095.0	5.35	2688.5	9678.4	5.65	2842.0	10,231.3
900	5.04	3203.3	11,532.0	5.40	3437.2	12,373.8	5.75	3657.3	13,166.1	6.08	3865.8	13,916.8
1000	5.37	4218.5	15,186.7	5.76	4526.0	16,293.4	6.13	4815.3	17,335.0	6.48	5089.4	18,321.7
1100	5.69	5410.5	19,477.9	6.11	5804.3	20,895.4	6.50	6174.8	22,229.1	6.87	6525.7	23,492.7
1200	6.00	6789.7	24,443.0	6.44	7283.2	26,219.3	6.85	7747.4	27,890.8	7.24	8187.2	29,474.1
1300	6.30	8366.0	30,117.5	6.76	8973.2	32,303.6	7.19	9544.6	34,360.5	7.60	10,085.8	36,308.7
1400	6.59	10,148.9	36,536.0	7.07	10,884.7	39,185.1	7.52	11,577.0	41,677.3	7.95	12,232.7	44,037.9
1500	6.87	12,147.7	43,731.7	7.37	13,027.6	46,899.3	7.84	13,855.3	49,879.2	8.28	14,639.3	52,701.6
1600	7.15	14,371.4	51,737.0	7.66	15,411.4	55,481.0	8.15	16,389.7	59,002.9	8.61	17,316.2	62,338.4

k(mm) = 0.05 T(°C) = 20

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.11	0.2	0.8	0.17	0.3	1.2	0.25	0.5	1.8	0.32	0.6	2.2
80	0.16	0.8	2.9	0.24	1.2	4.3	0.35	1.8	6.4	0.44	2.2	8.0
100	0.19	1.5	5.4	0.28	2.2	7.9	0.41	3.2	11.6	0.51	4.0	14.5
125	0.22	2.7	9.8	0.33	4.0	14.4	0.48	5.8	21.1	0.59	7.3	26.3
150	0.25	4.4	16.0	0.37	6.5	23.5	0.54	9.5	34.3	0.67	11.9	42.7
200	0.31	9.6	34.6	0.45	14.1	50.6	0.65	20.5	73.7	0.81	25.5	91.7
250	0.36	17.4	62.8	0.52	25.5	91.7	0.75	37.0	133.3	0.94	46.0	165.6
300	0.40	28.4	102.1	0.58	41.3	148.8	0.85	60.0	216.0	1.05	74.5	268.2
350	0.44	42.8	153.9	0.65	62.2	223.9	0.94	90.2	324.7	1.16	111.9	402.9
400	0.49	61.0	219.4	0.71	88.6	318.9	1.02	128.3	462.0	1.27	159.1	572.8
450	0.52	83.3	299.9	0.76	121.0	435.5	1.10	175.1	630.2	1.36	217.0	781.0
500	0.56	110.1	396.5	0.81	159.8	575.2	1.18	231.1	831.8	1.46	286.2	1030.4
600	0.63	178.4	642.2	0.91	258.5	930.5	1.32	373.3	1343.8	1.63	462.1	1663.5
700	0.70	268.0	964.9	1.01	387.9	1396.6	1.45	559.6	2014.7	1.80	692.4	2492.5
800	0.76	381.2	1372.3	1.10	551.2	1984.3	1.58	794.4	2860.0	1.95	982.4	3536.6
900	0.82	519.9	1871.5	1.18	751.1	2704.1	1.70	1081.8	3894.4	2.10	1337.1	4813.7
1000	0.87	686.0	2469.6	1.26	990.5	3565.7	1.82	1425.5	5131.9	2.24	1761.4	6340.9
1100	0.93	881.4	3173.0	1.34	1271.8	4578.5	1.92	1829.3	6585.7	2.38	2259.6	8134.6
1200	0.98	1107.8	3988.1	1.41	1597.6	5751.4	2.03	2296.8	8268.4	2.51	2836.2	10,210.2
1300	1.03	1366.9	4920.9	1.48	1970.3	7093.0	2.13	2831.2	10,192.3	2.63	3495.2	12,582.8
1400	1.08	1660.4	5977.3	1.55	2392.1	8611.7	2.23	3435.9	12,369.3	2.75	4240.8	15,266.7
1500	1.13	1989.7	7162.9	1.62	2865.4	10,315.4	2.33	4114.1	14,810.7	2.87	5076.7	18,276.1
1600	1.17	2356.5	8483.2	1.69	3392.3	12,212.1	2.42	4868.8	17,527.5	2.99	6006.8	21,624.5

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.37	0.7	2.6	0.42	0.8	3.0	0.47	0.9	3.3	0.51	1.0	3.6
80	0.51	2.6	9.3	0.58	2.9	10.5	0.64	3.2	11.6	0.70	3.5	12.6
100	0.60	4.7	16.9	0.68	5.3	19.1	0.75	5.9	21.1	0.81	6.4	22.9
125	0.69	8.5	30.7	0.78	9.6	34.6	0.86	10.6	38.2	0.94	11.5	41.5
150	0.78	13.8	49.8	0.88	15.6	56.2	0.97	17.2	62.0	1.06	18.7	67.3
200	0.95	29.7	107.0	1.07	33.5	120.5	1.17	36.9	132.8	1.27	40.0	144.1
250	1.09	53.6	193.1	1.23	60.4	217.3	1.35	66.5	239.4	1.47	72.1	259.7
300	1.23	86.8	312.4	1.38	97.7	351.6	1.52	107.5	387.1	1.65	116.6	419.8
350	1.35	130.3	469.1	1.52	146.6	527.7	1.68	161.3	580.8	1.82	174.9	629.7
400	1.47	185.2	666.7	1.66	208.3	749.8	1.82	229.2	825.0	1.98	248.4	894.4
450	1.59	252.5	908.8	1.78	283.8	1021.8	1.96	312.2	1124.1	2.13	338.4	1218.3
500	1.70	333.0	1198.7	1.91	374.3	1347.3	2.10	411.7	1482.0	2.27	446.1	1606.0
600	1.90	537.3	1934.1	2.14	603.7	2173.2	2.35	663.8	2389.7	2.54	719.2	2589.1
700	2.09	804.7	2896.8	2.35	903.9	3253.9	2.58	993.7	3577.2	2.80	1076.4	3875.0
800	2.27	1141.4	4108.9	2.55	1281.7	4614.2	2.80	1408.8	5071.8	3.04	1525.8	5493.0
900	2.44	1553.1	5591.1	2.74	1743.7	6277.5	3.01	1916.3	6898.8	3.26	2075.2	7470.8
1000	2.60	2045.4	7363.3	2.92	2296.0	8265.6	3.21	2522.9	9082.5	3.48	2731.8	9834.4
1100	2.76	2623.4	9444.1	3.10	2944.4	10,599.8	3.40	3235.0	11,645.8	3.69	3502.4	12,608.7
1200	2.91	3292.1	11,851.6	3.27	3694.4	13,300.0	3.59	4058.6	14,611.0	3.88	4393.8	15,817.5
1300	3.06	4056.4	14,603.0	3.43	4551.6	16,385.7	3.77	4999.7	17,999.1	4.08	5412.2	19,483.8
1400	3.20	4920.9	17,715.2	3.59	5521.0	19,875.6	3.94	6064.1	21,830.6	4.26	6563.8	23,629.8
1500	3.33	5890.1	21,204.3	3.74	6607.7	23,787.7	4.11	7257.1	26,125.4	4.44	7854.6	28,276.7
1600	3.47	6968.3	25,085.9	3.89	7816.5	28,139.5	4.27	8584.1	30,902.7	4.62	9290.4	33,445.4

k(mm) = 0.05 T(°C) = 20

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.54	1.1	3.9	0.58	1.1	4.1	0.62	1.2	4.4	0.68	1.3	4.8
80	0.75	3.8	13.6	0.80	4.0	14.5	0.85	4.3	15.3	0.93	4.7	16.9
100	0.87	6.8	24.6	0.93	7.3	26.2	0.98	7.7	27.7	1.08	8.5	30.6
125	1.01	12.4	44.5	1.07	13.2	47.4	1.14	13.9	50.2	1.25	15.4	55.3
150	1.14	20.1	72.2	1.21	21.4	76.9	1.28	22.6	81.3	1.41	24.9	89.6
200	1.37	43.0	154.6	1.45	45.7	164.6	1.54	48.3	174.0	1.69	53.2	191.5
250	1.58	77.4	278.6	1.68	82.3	296.4	1.77	87.0	313.3	1.95	95.7	344.7
300	1.77	125.1	450.3	1.88	133.0	479.0	1.99	140.6	506.1	2.19	154.6	556.7
350	1.95	187.6	675.3	2.07	199.5	718.2	2.19	210.8	758.8	2.41	231.8	834.4
400	2.12	266.4	959.0	2.25	283.3	1019.7	2.38	299.2	1077.3	2.62	329.0	1184.4
450	2.28	362.8	1306.2	2.43	385.8	1388.8	2.56	407.5	1466.9	2.82	447.9	1612.5
500	2.44	478.2	1721.6	2.59	508.4	1830.3	2.73	537.0	1933.1	3.01	590.2	2124.6
600	2.73	770.8	2774.8	2.90	819.3	2949.5	3.06	865.2	3114.7	3.36	950.7	3422.4
700	3.00	1153.4	4152.3	3.19	1225.8	4413.0	3.36	1294.4	4659.8	3.69	1422.0	5119.0
800	3.25	1634.8	5885.4	3.46	1737.3	6254.2	3.65	1834.2	6603.3	4.01	2014.7	7252.9
900	3.49	2223.2	8003.6	3.71	2362.3	8504.3	3.92	2493.9	8978.1	4.31	2738.9	9860.0
1000	3.73	2926.3	10,534.7	3.96	3109.1	11,192.9	4.18	3282.1	11,815.7	4.59	3604.1	12,974.7
1100	3.95	3751.5	13,505.4	4.19	3985.6	14,348.2	4.43	4207.1	15,145.6	4.86	4619.3	16,629.5
1200	4.16	4705.9	16,941.2	4.42	4999.2	17,997.2	4.67	5276.8	18,996.4	5.12	5793.2	20,855.7
1300	4.37	5796.3	20,866.7	4.64	6157.2	22,166.1	4.90	6498.8	23,395.5	5.37	7134.3	25,683.3
1400	4.57	7029.3	25,305.3	4.85	7466.6	26,879.8	5.12	7880.4	28,369.4	5.62	8650.4	31,141.3
1500	4.76	8411.2	30,280.2	5.06	8934.1	32,162.6	5.34	9428.8	33,943.7	5.86	10,349.4	37,257.8
1600	4.95	9948.1	35,813.3	5.26	10,566.2	38,038.2	5.55	11,150.9	40,143.2	6.09	12,238.8	44,059.8

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.74	1.5	5.2	0.79	1.6	5.6	0.85	1.7	6.0	0.90	1.8	6.3
80	1.01	5.1	18.3	1.09	5.5	19.7	1.16	5.8	21.0	1.23	6.2	22.2
100	1.17	9.2	33.2	1.26	9.9	35.6	1.34	10.5	37.9	1.42	11.1	40.1
125	1.36	16.7	60.0	1.46	17.9	64.4	1.55	19.0	68.5	1.64	20.1	72.4
150	1.53	27.0	97.2	1.64	29.0	104.3	1.74	30.8	110.9	1.84	32.6	117.2
200	1.84	57.7	207.6	1.97	61.9	222.7	2.09	65.8	236.8	2.21	69.5	250.2
250	2.11	103.8	373.6	2.27	111.3	400.6	2.41	118.3	426.0	2.55	125.0	450.0
300	2.37	167.6	603.3	2.54	179.6	646.7	2.70	191.0	687.5	2.85	201.7	726.1
350	2.61	251.1	904.1	2.80	269.2	969.0	2.97	286.1	1029.9	3.14	302.1	1087.7
400	2.84	356.4	1283.0	3.04	381.9	1374.8	3.23	405.9	1461.2	3.41	428.6	1542.9
450	3.05	485.1	1746.5	3.27	519.8	1871.3	3.47	552.4	1988.7	3.67	583.2	2099.7
500	3.26	639.1	2300.9	3.49	684.7	2465.1	3.71	727.6	2619.4	3.91	768.2	2765.4
600	3.64	1029.3	3705.6	3.90	1102.6	3969.3	4.14	1171.4	4217.2	4.37	1236.6	4451.7
700	4.00	1539.4	5541.7	4.28	1648.7	5935.3	4.55	1751.4	6305.2	4.80	1848.7	6655.2
800	4.34	2180.7	7850.6	4.65	2335.4	8407.4	4.94	2480.7	8930.5	5.21	2618.2	9425.4
900	4.66	2964.3	10,671.5	4.99	3174.2	11,427.1	5.30	3371.4	12,137.2	5.59	3558.0	12,808.9
1000	4.97	3900.3	14,041.1	5.32	4176.2	15,034.2	5.65	4435.3	15,967.2	5.96	4680.6	16,850.0
1100	5.26	4998.6	17,994.8	5.63	5351.7	19,266.2	5.98	5683.5	20,460.7	6.31	5997.4	21,590.7
1200	5.54	6268.4	22,566.4	5.93	6710.9	24,159.2	6.30	7126.6	25,655.7	6.65	7519.9	27,071.5
1300	5.82	7718.9	27,788.1	6.23	8263.3	29,747.9	6.61	8774.8	31,589.2	6.98	9258.6	33,331.0
1400	6.08	9358.7	33,691.4	6.51	10,018.3	36,065.8	6.91	10,637.9	38,296.5	7.29	11,224.1	40,406.8
1500	6.34	11,196.3	40,306.6	6.78	11,984.8	43,145.3	7.20	12,725.6	45,812.1	7.60	13,426.4	48,335.0
1600	6.58	13,239.7	47,663.0	7.05	14,171.6	51,017.7	7.48	15,047.0	54,169.3	7.90	15,875.2	57,150.7

k(mm) = 0.1 T(°C) = 20

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.11	0.2	0.8	0.17	0.3	1.2	0.25	0.5	1.7	0.31	0.6	2.2
80	0.16	0.8	2.9	0.23	1.2	4.2	0.34	1.7	6.2	0.43	2.1	7.7
100	0.19	1.5	5.3	0.27	2.1	7.7	0.40	3.1	11.2	0.49	3.9	14.0
125	0.22	2.7	9.6	0.32	3.9	14.0	0.46	5.7	20.4	0.57	7.0	25.4
150	0.25	4.4	15.7	0.36	6.3	22.8	0.52	9.2	33.2	0.65	11.4	41.2
200	0.30	9.4	33.8	0.44	13.7	49.2	0.63	19.8	71.3	0.78	24.5	88.4
250	0.35	17.0	61.4	0.50	24.7	89.1	0.73	35.8	128.8	0.90	44.3	159.4
300	0.39	27.7	99.7	0.57	40.1	144.5	0.82	57.9	208.5	1.01	71.7	258.0
350	0.43	41.7	150.1	0.63	60.4	217.3	0.90	87.0	313.2	1.12	107.6	387.3
400	0.47	59.4	213.9	0.68	85.9	309.2	0.98	123.7	445.4	1.22	152.9	550.4
450	0.51	81.2	292.2	0.74	117.2	422.0	1.06	168.7	607.3	1.31	208.4	750.1
500	0.55	107.3	386.1	0.79	154.8	557.2	1.13	222.6	801.2	1.40	274.8	989.3
600	0.61	173.6	624.9	0.88	250.2	900.6	1.27	359.3	1293.4	1.57	443.3	1596.0
700	0.68	260.6	938.2	0.97	375.2	1350.7	1.40	538.3	1937.9	1.73	663.9	2390.2
800	0.74	370.4	1333.5	1.06	532.8	1918.0	1.52	763.8	2749.7	1.87	941.7	3390.0
900	0.79	504.9	1817.7	1.14	725.7	2612.3	1.63	1039.6	3742.7	2.01	1281.3	4612.6
1000	0.85	665.9	2397.3	1.22	956.4	3443.2	1.74	1369.5	4930.1	2.15	1687.3	6074.3
1100	0.90	855.2	3078.9	1.29	1227.6	4419.4	1.85	1756.9	6324.8	2.28	2164.1	7790.6
1200	0.95	1074.5	3868.2	1.36	1541.6	5549.6	1.95	2205.2	7938.7	2.40	2715.7	9776.4
1300	1.00	1325.4	4771.3	1.43	1900.6	6842.1	2.05	2717.7	9783.6	2.52	3346.1	12,045.9
1400	1.05	1609.3	5793.6	1.50	2306.8	8304.6	2.14	3297.4	11,870.7	2.64	4059.1	14,612.9
1500	1.09	1928.0	6940.7	1.56	2762.5	9945.1	2.23	3947.5	14,210.8	2.75	4858.6	17,490.8
1600	1.14	2282.7	8217.8	1.63	3269.7	11,770.8	2.32	4670.8	16,814.7	2.86	5747.9	20,692.6

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.36	0.7	2.5	0.41	0.8	2.9	0.45	0.9	3.2	0.49	1.0	3.4
80	0.50	2.5	9.0	0.56	2.8	10.1	0.62	3.1	11.2	0.67	3.4	12.1
100	0.58	4.5	16.3	0.65	5.1	18.4	0.72	5.6	20.2	0.78	6.1	22.0
125	0.67	8.2	29.6	0.75	9.2	33.3	0.83	10.2	36.6	0.90	11.0	39.7
150	0.75	13.3	48.0	0.85	15.0	54.0	0.93	16.5	59.4	1.01	17.9	64.4
200	0.91	28.6	102.8	1.02	32.1	115.6	1.12	35.3	127.1	1.22	38.3	137.8
250	1.05	51.5	185.4	1.18	57.9	208.3	1.30	63.6	229.1	1.40	68.9	248.2
300	1.18	83.3	299.8	1.32	93.5	336.8	1.45	102.8	370.2	1.58	111.4	401.0
350	1.30	125.0	449.9	1.46	140.3	505.2	1.60	154.2	555.2	1.74	167.0	601.3
400	1.41	177.5	639.2	1.59	199.3	717.5	1.74	219.0	788.4	1.89	237.1	853.7
450	1.52	241.9	870.9	1.71	271.5	977.4	1.88	298.3	1073.8	2.03	322.9	1162.5
500	1.62	319.0	1148.2	1.82	357.9	1288.5	2.00	393.1	1415.3	2.17	425.6	1532.1
600	1.82	514.4	1851.7	2.04	577.0	2077.2	2.24	633.7	2281.2	2.43	685.8	2469.0
700	2.00	770.0	2772.2	2.24	863.6	3109.0	2.46	948.3	3413.8	2.67	1026.2	3694.2
800	2.17	1091.9	3930.7	2.44	1224.3	4407.5	2.67	1344.1	4838.8	2.89	1454.3	5235.6
900	2.33	1485.3	5347.1	2.62	1665.2	5994.7	2.87	1827.9	6580.6	3.11	1977.6	7119.5
1000	2.49	1955.6	7040.2	2.79	2192.2	7891.8	3.06	2406.1	8662.1	3.31	2603.0	9370.8
1100	2.64	2507.8	9027.9	2.96	2810.8	10,118.7	3.25	3084.8	11,105.4	3.51	3337.0	12,013.0
1200	2.78	3146.5	11,327.4	3.12	3526.3	12,694.7	3.42	3869.8	13,931.4	3.70	4185.8	15,069.1
1300	2.92	3876.4	13,955.1	3.27	4343.9	15,638.2	3.59	4766.8	17,160.4	3.88	5155.7	18,560.7
1400	3.05	4701.9	16,927.0	3.42	5268.6	18,966.9	3.76	5781.1	20,811.9	4.06	6252.5	22,509.0
1500	3.18	5627.4	20,258.5	3.57	6305.1	22,698.3	3.91	6918.0	24,904.7	4.23	7481.8	26,934.3
1600	3.31	6656.8	23,964.6	3.71	7458.0	26,848.9	4.07	8182.6	29,457.3	4.40	8849.0	31,856.6

k(mm) = 0.1 T(°C) = 20

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.52	1.0	3.7	0.56	1.1	3.9	0.59	1.2	4.2	0.65	1.3	4.6
80	0.72	3.6	13.0	0.77	3.8	13.9	0.81	4.1	14.6	0.89	4.5	16.1
100	0.83	6.5	23.6	0.89	7.0	25.1	0.94	7.4	26.5	1.03	8.1	29.2
125	0.96	11.8	42.6	1.03	12.6	45.3	1.08	13.3	47.9	1.19	14.6	52.7
150	1.09	19.2	69.1	1.15	20.4	73.5	1.22	21.6	77.6	1.34	23.7	85.4
200	1.31	41.0	147.7	1.39	43.6	157.1	1.47	46.1	165.9	1.61	50.6	182.3
250	1.51	73.9	266.0	1.60	78.5	282.7	1.69	82.9	298.6	1.86	91.1	328.0
300	1.69	119.4	429.7	1.79	126.8	456.6	1.89	133.9	482.1	2.08	147.1	529.6
350	1.86	178.9	644.2	1.98	190.1	684.5	2.09	200.7	722.6	2.29	220.4	793.6
400	2.02	254.0	914.4	2.15	269.9	971.5	2.27	284.9	1025.6	2.49	312.8	1126.1
450	2.17	345.9	1245.2	2.31	367.4	1322.8	2.44	387.9	1396.3	2.68	425.8	1532.9
500	2.32	455.8	1640.8	2.47	484.2	1743.0	2.60	511.0	1839.7	2.86	561.0	2019.5
600	2.60	734.4	2643.8	2.76	780.0	2808.0	2.91	823.2	2963.4	3.20	903.5	3252.5
700	2.85	1098.7	3955.3	3.03	1166.8	4200.6	3.20	1231.3	4432.6	3.51	1351.2	4864.3
800	3.10	1557.0	5605.1	3.29	1653.4	5952.2	3.47	1744.6	6280.6	3.81	1914.3	6891.5
900	3.33	2117.0	7621.3	3.53	2248.0	8092.7	3.73	2371.9	8538.7	4.09	2602.3	9368.3
1000	3.55	2786.3	10,030.5	3.77	2958.4	10,650.3	3.97	3121.3	11,236.7	4.36	3424.3	12,327.4
1100	3.76	3571.7	12,858.1	3.99	3792.2	13,651.9	4.21	4000.8	14,402.8	4.62	4388.8	15,799.8
1200	3.96	4480.1	16,128.2	4.21	4756.4	17,123.1	4.44	5017.9	18,064.3	4.87	5504.2	19,815.2
1300	4.16	5517.9	19,864.3	4.41	5858.0	21,088.9	4.66	6179.8	22,247.3	5.11	6778.4	24,402.2
1400	4.35	6691.4	24,088.9	4.61	7103.6	25,573.0	4.87	7493.6	26,976.9	5.34	8219.0	29,588.5
1500	4.53	8006.6	28,823.8	4.81	8499.6	30,598.7	5.07	8966.0	32,277.6	5.56	9833.6	35,400.8
1600	4.71	9469.5	34,090.2	5.00	10,052.3	36,188.3	5.27	10,603.6	38,173.0	5.78	11,629.1	41,864.9

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.71	1.4	5.0	0.76	1.5	5.4	0.81	1.6	5.7	0.85	1.7	6.0
80	0.97	4.9	17.5	1.04	5.2	18.8	1.10	5.5	20.0	1.17	5.9	21.1
100	1.12	8.8	31.6	1.20	9.4	33.9	1.28	10.0	36.1	1.35	10.6	38.1
125	1.29	15.9	57.1	1.39	17.0	61.2	1.47	18.1	65.1	1.56	19.1	68.7
150	1.45	25.7	92.5	1.56	27.5	99.1	1.66	29.3	105.3	1.75	30.9	111.2
200	1.75	54.8	197.5	1.87	58.8	211.5	1.99	62.4	224.8	2.10	65.9	237.3
250	2.01	98.7	355.1	2.15	105.7	380.4	2.29	112.3	404.1	2.41	118.5	426.6
300	2.25	159.2	573.3	2.41	170.5	613.9	2.56	181.1	652.1	2.70	191.2	688.3
350	2.48	238.6	858.9	2.66	255.5	919.7	2.82	271.3	976.8	2.98	286.4	1030.9
400	2.69	338.5	1218.6	2.88	362.4	1304.8	3.06	384.9	1385.7	3.23	406.2	1462.2
450	2.90	460.7	1658.7	3.10	493.3	1775.8	3.29	523.8	1885.8	3.48	552.7	1989.8
500	3.09	606.9	2185.0	3.31	649.7	2339.0	3.51	689.9	2483.8	3.71	728.0	2620.7
600	3.46	977.4	3518.5	3.70	1046.1	3766.1	3.93	1110.7	3998.7	4.14	1171.9	4218.7
700	3.80	1461.5	5261.5	4.06	1564.2	5631.2	4.32	1660.7	5978.5	4.55	1752.0	6307.1
800	4.12	2070.4	7453.4	4.41	2215.7	7976.6	4.68	2352.2	8468.0	4.94	2481.3	8932.9
900	4.42	2814.3	10,131.4	4.73	3011.6	10,841.8	5.03	3197.0	11,509.2	5.30	3372.3	12,140.4
1000	4.71	3703.0	13,330.7	5.05	3962.4	14,264.6	5.36	4206.1	15,141.9	5.65	4436.6	15,971.8
1100	4.99	4745.7	17,084.6	5.34	5078.0	18,280.8	5.67	5390.1	19,404.3	5.98	5685.3	20,467.0
1200	5.26	5951.5	21,425.5	5.63	6367.9	22,924.6	5.98	6759.1	24,332.7	6.30	7129.1	25,664.6
1300	5.52	7328.9	26,384.2	5.91	7841.4	28,229.2	6.27	8322.8	29,962.2	6.61	8778.2	31,601.5
1400	5.77	8886.2	31,990.5	6.18	9507.3	34,226.4	6.56	10,090.7	36,326.6	6.91	10,642.6	38,313.2
1500	6.02	10,631.5	38,273.2	6.44	11,374.2	40,947.1	6.83	12,071.9	43,458.7	7.20	12,731.8	45,834.4
1600	6.25	12,572.3	45,260.4	6.69	13,450.3	48,421.2	7.10	14,275.0	51,390.1	7.49	15,055.1	54,198.3

k(mm) = 0.5 T(°C) = 20

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.10	0.2	0.7	0.15	0.3	1.0	0.21	0.4	1.5	0.26	0.5	1.9
80	0.14	0.7	2.6	0.21	1.0	3.7	0.30	1.5	5.4	0.37	1.8	6.6
100	0.17	1.3	4.7	0.24	1.9	6.8	0.35	2.7	9.8	0.43	3.3	12.0
125	0.19	2.4	8.6	0.28	3.4	12.4	0.40	4.9	17.7	0.49	6.1	21.8
150	0.22	3.9	14.0	0.32	5.6	20.1	0.45	8.0	28.8	0.56	9.9	35.5
200	0.27	8.4	30.2	0.38	12.0	43.3	0.55	17.2	61.9	0.67	21.1	76.1
250	0.31	15.2	54.8	0.44	21.8	78.4	0.63	31.0	111.8	0.78	38.2	137.4
300	0.35	24.7	88.9	0.50	35.3	127.0	0.71	50.3	181.0	0.87	61.8	222.5
350	0.39	37.1	133.7	0.55	53.0	190.9	0.79	75.5	272.0	0.96	92.8	334.2
400	0.42	52.9	190.4	0.60	75.5	271.7	0.85	107.4	386.8	1.05	132.0	475.1
450	0.45	72.2	259.9	0.65	103.0	370.6	0.92	146.5	527.5	1.13	180.0	647.9
500	0.49	95.3	343.2	0.69	135.9	489.3	0.98	193.3	696.0	1.21	237.4	854.7
600	0.55	154.2	555.0	0.78	219.6	790.6	1.10	312.2	1124.1	1.36	383.3	1380.0
700	0.60	231.3	832.8	0.86	329.3	1185.6	1.22	468.0	1684.9	1.49	574.5	2068.2
800	0.65	328.6	1183.1	0.93	467.6	1683.5	1.32	664.4	2391.7	1.62	815.3	2935.2
900	0.70	447.8	1612.1	1.00	637.0	2293.1	1.42	904.6	3256.7	1.74	1110.1	3996.3
1000	0.75	590.5	2125.6	1.07	839.6	3022.5	1.52	1192.1	4291.6	1.86	1462.7	5265.6
1100	0.80	758.1	2729.3	1.13	1077.7	3879.8	1.61	1529.9	5507.6	1.98	1876.9	6757.0
1200	0.84	952.3	3428.4	1.20	1353.4	4872.4	1.70	1921.0	6915.4	2.08	2356.5	8483.4
1300	0.88	1174.5	4228.1	1.26	1668.8	6007.7	1.78	2368.2	8525.4	2.19	2904.9	10,457.7
1400	0.93	1426.0	5133.4	1.32	2025.7	7292.6	1.87	2874.3	10,347.4	2.29	3525.5	12,691.9
1500	0.97	1708.1	6149.2	1.37	2426.2	8734.2	1.95	3442.0	12,391.2	2.39	4221.6	15,197.8
1600	1.01	2022.2	7280.1	1.43	2871.9	10,338.8	2.03	4073.9	14,666.1	2.48	4996.4	17,987.0

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.31	0.6	2.2	0.35	0.7	2.4	0.38	0.7	2.7	0.41	0.8	2.9
80	0.42	2.1	7.7	0.48	2.4	8.6	0.52	2.6	9.5	0.57	2.8	10.3
100	0.49	3.9	14.0	0.55	4.3	15.7	0.61	4.8	17.2	0.66	5.2	18.6
125	0.57	7.0	25.3	0.64	7.9	28.4	0.70	8.6	31.1	0.76	9.4	33.7
150	0.65	11.4	41.1	0.72	12.8	46.1	0.79	14.0	50.5	0.86	15.2	54.7
200	0.78	24.5	88.2	0.87	27.4	98.8	0.96	30.1	108.3	1.04	32.5	117.1
250	0.90	44.2	159.1	1.01	49.5	178.2	1.11	54.3	195.4	1.20	58.7	211.3
300	1.01	71.5	257.5	1.13	80.1	288.3	1.24	87.8	316.2	1.34	95.0	341.9
350	1.12	107.4	386.7	1.25	120.3	432.9	1.37	131.9	474.7	1.48	142.5	513.2
400	1.22	152.7	549.7	1.36	170.9	615.3	1.49	187.4	674.7	1.61	202.6	729.3
450	1.31	208.2	749.4	1.47	233.0	838.8	1.61	255.5	919.7	1.74	276.1	994.1
500	1.40	274.6	988.6	1.57	307.4	1106.5	1.72	337.0	1213.2	1.85	364.2	1311.2
600	1.57	443.3	1595.9	1.75	496.1	1786.1	1.92	543.9	1958.0	2.08	587.8	2116.1
700	1.73	664.3	2391.4	1.93	743.4	2676.1	2.12	814.9	2933.5	2.29	880.6	3170.3
800	1.88	942.6	3393.5	2.10	1054.8	3797.3	2.30	1156.2	4162.4	2.49	1249.5	4498.1
900	2.02	1283.3	4619.8	2.26	1435.9	5169.3	2.47	1573.9	5666.0	2.67	1700.8	6122.8
1000	2.15	1690.8	6086.8	2.41	1891.8	6810.4	2.64	2073.5	7464.6	2.85	2240.6	8066.2
1100	2.28	2169.5	7810.3	2.55	2427.3	8738.4	2.80	2660.4	9577.5	3.03	2874.8	10,349.1
1200	2.41	2723.7	9805.5	2.69	3047.3	10,970.2	2.95	3339.8	12,023.3	3.19	3608.8	12,991.7
1300	2.53	3357.4	12,086.8	2.83	3756.1	13,522.1	3.10	4116.6	14,819.8	3.35	4448.1	16,013.2
1400	2.65	4074.6	14,668.4	2.96	4558.3	16,409.9	3.25	4995.7	17,984.4	3.51	5397.8	19,432.3
1500	2.76	4878.9	17,564.0	3.09	5458.0	19,648.8	3.38	5981.6	21,533.6	3.66	6463.0	23,266.9
1600	2.87	5774.1	20,786.8	3.21	6459.3	23,253.6	3.52	7078.8	25,483.8	3.80	7648.5	27,534.7

k(mm) = 0.5 T(°C) = 20

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.44	0.9	3.1	0.47	0.9	3.3	0.49	1.0	3.5	0.54	1.1	3.8
80	0.61	3.0	11.0	0.64	3.2	11.7	0.68	3.4	12.3	0.75	3.7	13.5
100	0.70	5.5	19.9	0.75	5.9	21.1	0.79	6.2	22.3	0.87	6.8	24.5
125	0.82	10.0	36.0	0.87	10.6	38.3	0.91	11.2	40.4	1.00	12.3	44.3
150	0.92	16.2	58.5	0.98	17.3	62.1	1.03	18.2	65.5	1.13	20.0	71.9
200	1.11	34.8	125.4	1.18	37.0	133.1	1.24	39.0	140.3	1.36	42.8	153.9
250	1.28	62.8	226.1	1.36	66.7	239.9	1.43	70.3	253.1	1.57	77.1	277.5
300	1.44	101.6	365.7	1.53	107.8	388.1	1.61	113.7	409.3	1.76	124.7	448.8
350	1.58	152.5	549.0	1.68	161.8	582.6	1.77	170.7	614.4	1.94	187.1	673.5
400	1.72	216.7	780.1	1.83	230.0	827.9	1.93	242.5	873.0	2.12	265.8	957.0
450	1.86	295.4	1063.3	1.97	313.4	1128.4	2.08	330.5	1189.9	2.28	362.3	1304.3
500	1.98	389.6	1402.5	2.11	413.4	1488.2	2.22	435.9	1569.3	2.43	477.8	1720.1
600	2.22	628.7	2263.3	2.36	667.1	2401.5	2.49	703.4	2532.3	2.73	771.0	2775.5
700	2.45	941.8	3390.6	2.60	999.3	3597.6	2.74	1053.7	3793.4	3.00	1154.9	4157.5
800	2.66	1336.3	4810.6	2.82	1417.8	5104.2	2.97	1494.9	5381.8	3.26	1638.4	5898.1
900	2.86	1818.9	6548.0	3.03	1929.8	6947.4	3.20	2034.7	7325.1	3.51	2229.9	8027.7
1000	3.05	2396.1	8626.1	3.24	2542.2	9152.1	3.41	2680.4	9649.5	3.74	2937.4	10,574.7
1100	3.23	3074.3	11,067.3	3.43	3261.6	11,741.9	3.62	3438.9	12,380.0	3.97	3768.5	13,566.7
1200	3.41	3859.2	13,893.1	3.62	4094.4	14,739.7	3.82	4316.8	15,540.5	4.18	4730.5	17,029.8
1300	3.58	4756.7	17,124.0	3.80	5046.5	18,167.3	4.01	5320.6	19,154.0	4.39	5830.4	20,989.3
1400	3.75	5772.2	20,779.9	3.98	6123.8	22,045.7	4.19	6456.4	23,242.9	4.60	7074.9	25,469.7
1500	3.91	6911.2	24,880.3	4.15	7332.1	26,395.5	4.37	7730.2	27,828.8	4.79	8470.7	30,494.4
1600	4.07	8178.8	29,443.6	4.32	8676.8	31,236.6	4.55	9147.9	32,932.4	4.99	10,024.0	36,086.5

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.59	1.2	4.2	0.63	1.2	4.4	0.67	1.3	4.7	0.70	1.4	5.0
80	0.81	4.1	14.6	0.86	4.3	15.6	0.92	4.6	16.6	0.97	4.9	17.5
100	0.94	7.4	26.5	1.00	7.9	28.3	1.06	8.4	30.1	1.12	8.8	31.7
125	1.08	13.3	47.9	1.16	14.2	51.2	1.23	15.1	54.4	1.30	15.9	57.4
150	1.22	21.6	77.7	1.31	23.1	83.1	1.39	24.5	88.2	1.46	25.8	93.0
200	1.47	46.2	166.4	1.57	49.4	178.0	1.67	52.5	188.9	1.76	55.3	199.2
250	1.70	83.3	299.9	1.82	89.1	320.8	1.93	94.6	340.4	2.03	99.7	359.0
300	1.91	134.7	485.0	2.04	144.1	518.8	2.16	152.9	550.5	2.28	161.3	580.5
350	2.10	202.2	727.9	2.25	216.3	778.5	2.39	229.5	826.1	2.51	242.0	871.1
400	2.29	287.3	1034.3	2.45	307.3	1106.1	2.59	326.0	1173.7	2.74	343.8	1237.5
450	2.46	391.5	1409.5	2.63	418.7	1507.4	2.79	444.3	1599.4	2.95	468.5	1686.4
500	2.63	516.3	1858.8	2.81	552.2	1987.9	2.98	585.9	2109.2	3.15	617.7	2223.9
600	2.95	833.1	2999.1	3.15	890.9	3207.3	3.34	945.2	3402.8	3.52	996.6	3587.8
700	3.24	1247.9	4492.3	3.47	1334.4	4804.0	3.68	1415.8	5096.7	3.88	1492.7	5373.6
800	3.52	1770.3	6373.0	3.77	1893.0	6814.9	4.00	2008.4	7230.1	4.21	2117.4	7622.7
900	3.79	2409.4	8673.7	4.05	2576.4	9275.1	4.30	2733.3	9839.9	4.53	2881.7	10,374.1
1000	4.04	3173.8	11,425.6	4.32	3393.8	12,217.5	4.58	3600.4	12,961.3	4.83	3795.8	13,664.8
1100	4.28	4071.7	14,658.0	4.58	4353.8	15,673.8	4.86	4618.8	16,627.8	5.12	4869.5	17,530.2
1200	4.52	5111.0	18,399.4	4.83	5465.1	19,674.3	5.13	5797.7	20,871.6	5.40	6112.2	22,004.1
1300	4.75	6299.2	22,677.1	5.07	6735.6	24,248.1	5.38	7145.4	25,723.5	5.68	7533.1	27,119.1
1400	4.97	7643.7	27,517.4	5.31	8173.2	29,423.4	5.63	8670.4	31,213.5	5.94	9140.8	32,906.7
1500	5.18	9151.6	32,945.8	5.54	9785.4	35,227.5	5.87	10,380.7	37,370.5	6.19	10,943.7	39,397.5
1600	5.39	10,829.7	38,987.1	5.76	11,579.7	41,686.9	6.11	12,284.0	44,222.6	6.44	12,950.3	46,620.9

k(mm) = 1 T(°C) = 20

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.09	0.2	0.7	0.13	0.3	1.0	0.19	0.4	1.4	0.24	0.5	1.7
80	0.13	0.7	2.4	0.19	0.9	3.4	0.27	1.4	4.9	0.33	1.7	6.0
100	0.15	1.2	4.3	0.22	1.7	6.2	0.31	2.5	8.9	0.39	3.0	10.9
125	0.18	2.2	7.9	0.26	3.1	11.3	0.37	4.5	16.2	0.45	5.5	19.9
150	0.20	3.6	12.9	0.29	5.1	18.5	0.41	7.3	26.3	0.51	9.0	32.3
200	0.25	7.8	27.9	0.35	11.1	39.8	0.50	15.7	56.6	0.62	19.3	69.6
250	0.29	14.1	50.6	0.41	20.0	72.1	0.58	28.5	102.5	0.71	35.0	125.8
300	0.32	22.8	82.2	0.46	32.5	117.0	0.65	46.2	166.2	0.80	56.7	204.0
350	0.36	34.4	123.8	0.51	48.9	176.1	0.72	69.5	250.0	0.89	85.2	306.8
400	0.39	49.0	176.3	0.55	69.6	250.7	0.79	98.9	355.9	0.97	121.3	436.7
450	0.42	66.9	240.8	0.60	95.1	342.3	0.85	135.0	485.9	1.04	165.6	596.0
500	0.45	88.4	318.2	0.64	125.6	452.1	0.91	178.2	641.6	1.11	218.6	787.1
600	0.51	143.1	515.0	0.72	203.2	731.4	1.02	288.2	1037.6	1.25	353.5	1272.6
700	0.56	214.8	773.3	0.79	305.0	1097.9	1.12	432.5	1557.1	1.38	530.4	1909.4
800	0.61	305.4	1099.3	0.86	433.4	1560.3	1.22	614.5	2212.3	1.50	753.5	2712.7
900	0.65	416.3	1498.8	0.93	590.7	2126.7	1.32	837.5	3014.9	1.61	1026.8	3696.5
1000	0.70	549.2	1977.2	0.99	779.2	2805.0	1.41	1104.4	3975.9	1.72	1354.0	4874.4
1100	0.74	705.5	2539.8	1.05	1000.7	3602.6	1.49	1418.3	5105.7	1.83	1738.7	6259.2
1200	0.78	886.6	3191.8	1.11	1257.4	4526.6	1.58	1781.8	6414.6	1.93	2184.3	7863.4
1300	0.82	1093.8	3937.8	1.17	1551.1	5583.9	1.66	2197.8	7912.2	2.03	2694.1	9698.9
1400	0.86	1328.5	4782.7	1.22	1883.7	6781.2	1.73	2668.9	9607.9	2.13	3271.4	11,777.1
1500	0.90	1591.9	5731.0	1.28	2256.9	8124.9	1.81	3197.5	11,510.9	2.22	3919.2	14,109.1
1600	0.94	1885.3	6787.1	1.33	2672.6	9621.3	1.88	3786.1	13,629.9	2.31	4640.5	16,705.9

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.28	0.5	2.0	0.31	0.6	2.2	0.34	0.7	2.4	0.37	0.7	2.6
80	0.38	1.9	7.0	0.43	2.2	7.8	0.47	2.4	8.5	0.51	2.6	9.2
100	0.45	3.5	12.7	0.50	3.9	14.2	0.55	4.3	15.5	0.59	4.7	16.8
125	0.52	6.4	23.0	0.58	7.2	25.7	0.64	7.8	28.2	0.69	8.5	30.5
150	0.59	10.4	37.4	0.66	11.6	41.9	0.72	12.8	45.9	0.78	13.8	49.6
200	0.71	22.3	80.5	0.80	25.0	90.1	0.87	27.4	98.7	0.94	29.6	106.7
250	0.82	40.4	145.5	0.92	45.2	162.8	1.01	49.6	178.5	1.09	53.6	192.9
300	0.93	65.5	235.9	1.04	73.3	264.0	1.14	80.4	289.4	1.23	86.9	312.7
350	1.02	98.5	354.7	1.15	110.2	396.9	1.26	120.8	435.0	1.36	130.6	470.1
400	1.12	140.2	504.8	1.25	156.9	564.8	1.37	172.0	619.1	1.48	185.8	668.9
450	1.20	191.4	688.9	1.35	214.1	770.8	1.48	234.7	844.8	1.59	253.6	912.8
500	1.29	252.7	909.7	1.44	282.7	1017.7	1.58	309.8	1115.3	1.70	334.8	1205.2
600	1.44	408.5	1470.7	1.62	457.0	1645.2	1.77	500.8	1803.0	1.91	541.1	1948.1
700	1.59	612.9	2206.5	1.78	685.6	2468.2	1.95	751.3	2704.8	2.11	811.8	2922.4
800	1.73	870.7	3134.5	1.94	973.9	3506.1	2.12	1067.3	3842.1	2.29	1153.1	4151.1
900	1.86	1186.4	4271.1	2.09	1327.0	4777.4	2.29	1454.2	5235.1	2.47	1571.1	5656.0
1000	1.99	1564.4	5631.9	2.23	1749.8	6299.3	2.44	1917.4	6902.6	2.64	2071.5	7457.5
1100	2.11	2008.8	7231.7	2.36	2246.8	8088.5	2.59	2462.0	8863.1	2.80	2659.8	9575.4
1200	2.23	2523.6	9084.9	2.50	2822.5	10,161.0	2.73	3092.7	11,133.9	2.95	3341.3	12,028.6
1300	2.34	3112.5	11,205.1	2.62	3481.2	12,532.2	2.87	3814.4	13,731.9	3.10	4120.9	14,835.2
1400	2.46	3779.4	13,605.8	2.75	4226.9	15,216.9	3.01	4631.5	16,673.5	3.25	5003.6	18,012.9
1500	2.56	4527.7	16,299.6	2.87	5063.7	18,229.5	3.14	5548.4	19,974.2	3.39	5994.1	21,578.7
1600	2.67	5360.9	19,299.2	2.98	5995.5	21,584.0	3.27	6569.3	23,649.5	3.53	7097.0	25,549.0

k(mm) = 1 T(°C) = 20

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.39	0.8	2.8	0.42	0.8	3.0	0.44	0.9	3.1	0.48	1.0	3.4
80	0.55	2.7	9.9	0.58	2.9	10.5	0.61	3.1	11.1	0.67	3.4	12.1
100	0.64	5.0	18.0	0.67	5.3	19.1	0.71	5.6	20.1	0.78	6.1	22.1
125	0.74	9.1	32.7	0.78	9.6	34.7	0.83	10.2	36.5	0.91	11.1	40.1
150	0.83	14.7	53.1	0.89	15.7	56.3	0.93	16.5	59.4	1.02	18.1	65.1
200	1.01	31.7	114.1	1.07	33.6	121.1	1.13	35.5	127.7	1.24	38.9	140.0
250	1.17	57.3	206.3	1.24	60.8	218.9	1.31	64.1	230.9	1.43	70.3	253.0
300	1.31	92.9	334.4	1.39	98.6	354.8	1.47	103.9	374.1	1.61	113.9	410.0
350	1.45	139.7	502.7	1.54	148.2	533.4	1.62	156.2	562.4	1.78	171.2	616.3
400	1.58	198.7	715.4	1.68	210.8	759.0	1.77	222.3	800.2	1.94	243.6	877.0
450	1.70	271.2	976.2	1.81	287.7	1035.6	1.91	303.3	1091.9	2.09	332.4	1196.6
500	1.82	358.0	1288.7	1.93	379.8	1367.3	2.04	400.4	1441.5	2.23	438.8	1579.6
600	2.05	578.7	2083.2	2.17	613.9	2210.0	2.29	647.2	2330.0	2.51	709.2	2553.1
700	2.26	868.0	3125.0	2.39	920.9	3315.2	2.52	970.9	3495.1	2.76	1063.8	3829.7
800	2.45	1233.0	4438.7	2.60	1308.0	4708.9	2.74	1379.0	4964.4	3.01	1511.0	5439.6
900	2.64	1679.9	6047.7	2.80	1782.1	6415.7	2.95	1878.8	6763.7	3.24	2058.6	7411.0
1000	2.82	2215.0	7973.9	2.99	2349.7	8459.0	3.15	2477.2	8917.8	3.46	2714.2	9771.1
1100	2.99	2844.0	10,238.4	3.17	3017.0	10,861.1	3.35	3180.6	11,450.1	3.67	3484.9	12,545.6
1200	3.16	3572.6	12,861.3	3.35	3789.9	13,643.5	3.53	3995.3	14,383.2	3.87	4377.5	15,759.1
1300	3.32	4406.2	15,862.2	3.52	4674.1	16,826.7	3.71	4927.5	17,738.9	4.07	5398.8	19,435.7
1400	3.48	5349.9	19,259.7	3.69	5675.2	20,430.7	3.89	5982.8	21,538.2	4.26	6555.0	23,598.1
1500	3.63	6408.9	23,072.1	3.85	6798.5	24,474.7	4.06	7167.0	25,801.4	4.44	7852.5	28,268.9
1600	3.77	7588.1	27,317.1	4.00	8049.3	28,977.6	4.22	8485.6	30,548.2	4.62	9297.1	33,469.4

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.52	1.0	3.7	0.56	1.1	4.0	0.59	1.2	4.2	0.63	1.2	4.4
80	0.73	3.6	13.1	0.78	3.9	14.0	0.82	4.1	14.9	0.87	4.4	15.7
100	0.84	6.6	23.9	0.90	7.1	25.5	0.96	7.5	27.1	1.01	7.9	28.6
125	0.98	12.0	43.3	1.05	12.9	46.3	1.11	13.7	49.1	1.17	14.4	51.8
150	1.11	19.6	70.4	1.18	20.9	75.3	1.26	22.2	79.9	1.32	23.4	84.2
200	1.34	42.0	151.3	1.43	44.9	161.8	1.52	47.7	171.7	1.60	50.3	181.0
250	1.55	75.9	273.4	1.65	81.2	292.4	1.76	86.2	310.2	1.85	90.8	327.1
300	1.74	123.1	443.0	1.86	131.6	473.8	1.98	139.6	502.6	2.08	147.2	529.9
350	1.92	185.0	665.9	2.06	197.8	712.1	2.18	209.9	755.5	2.30	221.2	796.5
400	2.09	263.2	947.5	2.24	281.4	1013.2	2.38	298.6	1074.8	2.50	314.8	1133.2
450	2.26	359.1	1292.8	2.41	384.0	1382.4	2.56	407.4	1466.5	2.70	429.5	1546.1
500	2.41	474.1	1706.7	2.58	506.9	1824.9	2.74	537.8	1935.9	2.89	566.9	2040.9
600	2.71	766.2	2758.4	2.90	819.3	2949.4	3.07	869.1	3128.8	3.24	916.2	3298.5
700	2.99	1149.3	4137.5	3.19	1228.9	4423.9	3.39	1303.6	4692.9	3.57	1374.3	4947.4
800	3.25	1632.4	5876.6	3.47	1745.4	6283.3	3.68	1851.5	6665.3	3.88	1951.8	7026.6
900	3.50	2224.0	8006.3	3.74	2377.9	8560.3	3.96	2522.4	9080.7	4.18	2659.1	9572.9
1000	3.73	2932.2	10,555.8	3.99	3135.0	11,286.2	4.23	3325.6	11,972.2	4.46	3505.8	12,621.0
1100	3.96	3764.7	13,552.9	4.24	4025.2	14,490.6	4.49	4269.8	15,371.3	4.74	4501.2	16,204.2
1200	4.18	4729.0	17,024.4	4.47	5056.1	18,202.1	4.74	5363.4	19,308.3	5.00	5654.0	20,354.5
1300	4.39	5832.2	20,996.0	4.70	6235.6	22,448.3	4.98	6614.5	23,812.3	5.25	6972.9	25,102.5
1400	4.60	7081.2	25,492.5	4.92	7571.0	27,255.7	5.22	8031.0	28,911.7	5.50	8466.1	30,478.0
1500	4.80	8482.8	30,537.9	5.13	9069.4	32,650.0	5.44	9620.4	34,633.6	5.74	10,141.6	36,509.8
1600	5.00	10,043.3	36,155.8	5.34	10,737.8	38,656.2	5.67	11,390.2	41,004.6	5.97	12,007.2	43,225.8

k(mm) = 5 T(°C) = 20

D	S =	0.0005		S =	0.001		S =	0.002		S =	0.003	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.07	0.1	0.5	0.10	0.2	0.7	0.14	0.3	1.0	0.17	0.3	1.2
80	0.10	0.5	1.8	0.14	0.7	2.5	0.20	1.0	3.6	0.24	1.2	4.4
100	0.12	0.9	3.3	0.16	1.3	4.6	0.23	1.8	6.6	0.29	2.2	8.1
125	0.14	1.7	6.0	0.19	2.4	8.5	0.27	3.4	12.1	0.34	4.1	14.8
150	0.16	2.7	9.9	0.22	3.9	14.0	0.31	5.5	19.9	0.38	6.8	24.4
200	0.19	6.0	21.6	0.27	8.5	30.6	0.38	12.0	43.3	0.47	14.8	53.1
250	0.22	11.0	39.4	0.32	15.5	55.9	0.45	22.0	79.1	0.55	26.9	97.0
300	0.25	17.9	64.4	0.36	25.3	91.3	0.51	35.9	129.2	0.62	44.0	158.4
350	0.28	27.1	97.5	0.40	38.3	138.0	0.56	54.3	195.5	0.69	66.5	239.5
400	0.31	38.7	139.4	0.44	54.8	197.4	0.62	77.6	279.5	0.76	95.1	342.5
450	0.33	53.1	191.1	0.47	75.2	270.6	0.67	106.4	383.1	0.82	130.4	469.4
500	0.36	70.4	253.3	0.51	99.6	358.6	0.72	141.0	507.6	0.88	172.8	622.0
600	0.40	114.5	412.0	0.57	162.0	583.4	0.81	229.4	825.7	0.99	281.0	1011.6
700	0.45	172.6	621.3	0.63	244.3	879.6	0.90	345.8	1244.8	1.10	423.6	1525.1
800	0.49	246.2	886.3	0.69	348.5	1254.6	0.98	493.2	1775.5	1.20	604.2	2175.2
900	0.53	336.7	1212.0	0.75	476.5	1715.6	1.06	674.4	2427.7	1.30	826.1	2974.1
1000	0.57	445.3	1603.1	0.80	630.3	2269.0	1.14	891.9	3210.8	1.39	1092.6	3933.4
1100	0.60	573.4	2064.1	0.85	811.5	2921.4	1.21	1148.3	4133.8	1.48	1406.7	5064.1
1200	0.64	722.1	2599.4	0.90	1021.9	3678.8	1.28	1445.9	5205.4	1.57	1771.3	6376.7
1300	0.67	892.5	3213.1	0.95	1263.1	4547.2	1.35	1787.2	6433.9	1.65	2189.3	7881.6
1400	0.71	1085.9	3909.2	1.00	1536.7	5532.2	1.41	2174.3	7827.4	1.73	2663.5	9588.6
1500	0.74	1303.3	4691.8	1.04	1844.3	6639.4	1.48	2609.4	9393.8	1.81	3196.5	11,507.3
1600	0.77	1545.7	5564.5	1.09	2187.3	7874.2	1.54	3094.6	11,140.6	1.89	3790.8	13,647.0

D	S=	0.004		S=	0.005		S=	0.006		S=	0.007	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.20	0.4	1.4	0.22	0.4	1.5	0.24	0.5	1.7	0.26	0.5	1.8
80	0.28	1.4	5.1	0.31	1.6	5.7	0.34	1.7	6.2	0.37	1.9	6.7
100	0.33	2.6	9.3	0.37	2.9	10.4	0.40	3.2	11.4	0.44	3.4	12.3
125	0.39	4.8	17.1	0.43	5.3	19.2	0.48	5.8	21.0	0.51	6.3	22.7
150	0.44	7.8	28.1	0.49	8.7	31.5	0.54	9.6	34.5	0.59	10.4	37.3
200	0.54	17.0	61.4	0.61	19.1	68.6	0.66	20.9	75.2	0.72	22.6	81.2
250	0.63	31.1	112.0	0.71	34.8	125.3	0.78	38.1	137.2	0.84	41.2	148.3
300	0.72	50.8	182.9	0.80	56.8	204.5	0.88	62.3	224.1	0.95	67.3	242.1
350	0.80	76.8	276.6	0.89	85.9	309.4	0.98	94.1	338.9	1.06	101.7	366.1
400	0.87	109.9	395.6	0.98	122.9	442.4	1.07	134.6	484.7	1.16	145.4	523.6
450	0.95	150.6	542.1	1.06	168.4	606.2	1.16	184.5	664.2	1.25	199.3	717.5
500	1.02	199.6	718.4	1.14	223.1	803.3	1.25	244.5	880.1	1.34	264.1	950.7
600	1.15	324.5	1168.4	1.28	362.9	1306.5	1.41	397.6	1431.3	1.52	429.5	1546.1
700	1.27	489.3	1761.3	1.42	547.1	1969.5	1.56	599.4	2157.7	1.68	647.4	2330.8
800	1.39	697.8	2512.2	1.55	780.3	2809.0	1.70	854.8	3077.4	1.84	923.4	3324.2
900	1.50	954.1	3434.8	1.68	1066.9	3840.7	1.84	1168.8	4207.6	1.98	1262.5	4545.0
1000	1.61	1261.8	4542.6	1.80	1410.9	5079.3	1.97	1545.7	5564.6	2.13	1669.7	6010.8
1100	1.71	1624.5	5848.3	1.91	1816.5	6539.3	2.09	1990.0	7164.0	2.26	2149.6	7738.4
1200	1.81	2045.6	7364.2	2.02	2287.3	8234.2	2.22	2505.8	9020.8	2.39	2706.7	9744.1
1300	1.90	2528.4	9102.1	2.13	2827.0	10,177.3	2.33	3097.1	11,149.4	2.52	3345.4	12,043.4
1400	2.00	3075.9	11,073.3	2.23	3439.3	12,381.4	2.45	3767.8	13,564.0	2.64	4069.9	14,651.5
1500	2.09	3691.4	13,289.0	2.34	4127.4	14,858.8	2.56	4521.7	16,278.0	2.76	4884.2	17,583.1
1600	2.18	4377.8	15,760.0	2.43	4894.9	17,621.6	2.67	5362.4	19,304.6	2.88	5792.3	20,852.3

k(mm) = 5 T(°C) = 20

D	S=	0.008		S=	0.009		S=	0.010		S=	0.012	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.28	0.5	2.0	0.29	0.6	2.1	0.31	0.6	2.2	0.34	0.7	2.4
80	0.40	2.0	7.2	0.42	2.1	7.6	0.44	2.2	8.0	0.49	2.4	8.8
100	0.47	3.7	13.2	0.50	3.9	14.0	0.52	4.1	14.8	0.57	4.5	16.2
125	0.55	6.7	24.3	0.58	7.2	25.8	0.61	7.5	27.2	0.67	8.3	29.7
150	0.63	11.1	39.9	0.66	11.7	42.3	0.70	12.4	44.6	0.77	13.6	48.8
200	0.77	24.1	86.8	0.81	25.6	92.1	0.86	27.0	97.1	0.94	29.6	106.4
250	0.90	44.0	158.5	0.95	46.7	168.2	1.00	49.2	177.3	1.10	53.9	194.2
300	1.02	71.9	258.8	1.08	76.3	274.6	1.14	80.4	289.4	1.25	88.1	317.1
350	1.13	108.7	391.5	1.20	115.3	415.2	1.26	121.6	437.7	1.38	133.2	479.6
400	1.24	155.5	559.8	1.31	164.9	593.8	1.38	173.9	626.0	1.52	190.5	685.8
450	1.34	213.1	767.1	1.42	226.0	813.7	1.50	238.3	857.7	1.64	261.0	939.7
500	1.44	282.3	1016.4	1.53	299.5	1078.2	1.61	315.7	1136.5	1.76	345.9	1245.1
600	1.62	459.2	1653.0	1.72	487.0	1753.4	1.82	513.4	1848.3	1.99	562.5	2024.9
700	1.80	692.2	2491.8	1.91	734.2	2643.1	2.01	774.0	2786.2	2.20	847.9	3052.4
800	1.96	987.2	3553.9	2.08	1047.1	3769.7	2.20	1103.8	3973.8	2.41	1209.3	4353.3
900	2.12	1349.7	4859.1	2.25	1431.7	5154.1	2.37	1509.2	5433.1	2.60	1653.3	5952.0
1000	2.27	1785.0	6426.1	2.41	1893.4	6816.2	2.54	1995.9	7185.2	2.78	2186.5	7871.4
1100	2.42	2298.1	8273.1	2.56	2437.6	8775.3	2.70	2569.5	9250.3	2.96	2814.9	10,133.7
1200	2.56	2893.7	10,417.3	2.71	3069.3	11,049.6	2.86	3235.5	11,647.7	3.13	3544.5	12,760.0
1300	2.69	3576.5	12,875.5	2.86	3793.6	13,657.0	3.01	3998.9	14,396.1	3.30	4380.8	15,770.9
1400	2.83	4351.0	15,663.7	3.00	4615.1	16,614.4	3.16	4864.9	17,513.7	3.46	5329.5	19,186.1
1500	2.95	5221.6	18,797.8	3.13	5538.5	19,938.7	3.30	5838.3	21,017.8	3.62	6395.8	23,024.8
1600	3.08	6192.5	22,292.9	3.27	6568.3	23,645.9	3.44	6923.8	24,925.6	3.77	7584.9	27,305.7

D	S=	0.014		S=	0.016		S=	0.018		S=	0.020	
(mm)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)	v(m/s)	Q(l/s)	Q(m^3/h)
50	0.37	0.7	2.6	0.39	0.8	2.8	0.42	0.8	2.9	0.44	0.9	3.1
80	0.52	2.6	9.5	0.56	2.8	10.1	0.59	3.0	10.8	0.63	3.2	11.3
100	0.62	4.9	17.5	0.66	5.2	18.7	0.70	5.5	19.8	0.74	5.8	20.9
125	0.73	8.9	32.1	0.78	9.5	34.4	0.83	10.1	36.5	0.87	10.7	38.4
150	0.83	14.7	52.8	0.89	15.7	56.4	0.94	16.6	59.8	0.99	17.5	63.1
200	1.02	31.9	114.9	1.09	34.1	122.9	1.15	36.2	130.4	1.22	38.2	137.4
250	1.19	58.3	209.8	1.27	62.3	224.3	1.35	66.1	237.9	1.42	69.7	250.8
300	1.35	95.2	342.6	1.44	101.7	366.2	1.53	107.9	388.5	1.61	113.8	409.5
350	1.50	143.9	518.0	1.60	153.8	553.8	1.70	163.2	587.5	1.79	172.0	619.3
400	1.64	205.8	740.8	1.75	220.0	792.0	1.86	233.3	840.1	1.96	246.0	885.5
450	1.77	282.0	1015.1	1.90	301.4	1085.2	2.01	319.7	1151.1	2.12	337.1	1213.4
500	1.90	373.6	1345.0	2.03	399.4	1437.9	2.16	423.7	1525.2	2.27	446.6	1607.8
600	2.15	607.6	2187.2	2.30	649.5	2338.4	2.44	689.0	2480.3	2.57	726.3	2614.6
700	2.38	915.9	3297.1	2.54	979.1	3524.9	2.70	1038.6	3738.9	2.84	1094.8	3941.3
800	2.60	1306.2	4702.4	2.78	1396.5	5027.2	2.95	1481.2	5332.4	3.11	1561.4	5621.0
900	2.81	1785.9	6429.2	3.00	1909.3	6873.3	3.18	2025.1	7290.5	3.36	2134.7	7685.1
1000	3.01	2361.8	8502.5	3.21	2525.0	9089.8	3.41	2678.2	9641.5	3.59	2823.1	10,163.3
1100	3.20	3040.6	10,946.1	3.42	3250.6	11,702.2	3.63	3447.9	12,412.4	3.82	3634.5	13,084.1
1200	3.39	3828.6	13,782.9	3.62	4093.1	14,735.0	3.84	4341.5	15,629.3	4.05	4576.4	16,475.0
1300	3.57	4732.0	17,035.1	3.81	5058.9	18,211.9	4.04	5365.9	19,317.1	4.26	5656.2	20,362.4
1400	3.74	5756.7	20,724.1	4.00	6154.3	22,155.6	4.24	6527.8	23,500.1	4.47	6881.0	24,771.8
1500	3.91	6908.5	24,870.5	4.18	7385.7	26,588.4	4.43	7833.8	28,201.9	4.67	8257.8	29,727.9
1600	4.07	8192.9	29,494.5	4.36	8758.8	31,531.7	4.62	9290.3	33,445.2	4.87	9793.0	35,254.9

APPENDIX 5

Spreadsheet Hydraulic Lessons – Overview

UNESCO-IHE
Institute for Water Education

Version 1.4
October 2005

Spreadsheet Hydraulic Lessons in

Water Transport & Distribution
Part I

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Disclaimer

Introduction

The spreadsheet hydraulic lessons have been developed as an aid for steady state hydraulic calculations of simple water transport and distribution networks. These are to be carried out while solving the workshop problems that should normally be calculated manually; the spreadsheet serves here as a fast check of the results. Moreover, the spreadsheet lessons will help teachers to demonstrate a wider range of problems in a clear way, as well as to allow the students to continue analysing them at home. Ultimately, a real understanding of the hydraulic concepts will be reached through 'playing' with the data.

Over forty problems have been classified in eight groups/worksheets according to the contents of the book 'Introduction to Water Transport and Distribution' by N. Trifunović. This book covers a core curriculum of ' Water Transport and Distribution 1', which is a three-week module in the Water Supply Engineering specialisation at UNESCO-IHE. Outside the regular MSc programme, this module is also offered as a stand-alone short course/distance learning package.

To be able to use the spreadsheet, brief accompanying instructions for each exercise are given in the 'About' worksheet (see below).
Each layout covers approximately one full screen (30 rows) consisting of drawings, tables and graphs. In the tables:

The green colour indicates input cells. Except for the headers, these cells are unprotected and their contents are used for calculations.
The brown colour indicates output cells. These cells contain fixed formulae and are therefore protected.

Moreover, some intermediate calculations have been moved further to the right in the worksheet, being irrelevant for educational purposes.

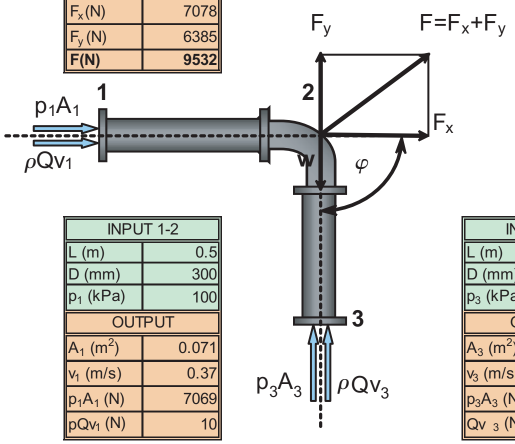
Each lesson serves as a kind of chess problem, in which 'check-mate' should be reached within a few, correct moves. This suggests a study process where thinking takes more time than the execution, which was the main concept in the development of the exercises. Any simplifications that have been introduced (neglected minor losses, pump curve definition, etc.) were meant to facilitate this process. In addition, the worksheets have been designed without complicated routines or macros; only a superficial knowledge of spreadsheets is required to be able to use them effectively.

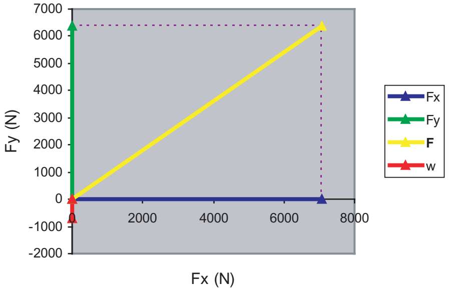
This is the first edition and any suggestions for improvement or extension are obviously welcome.

N. Trifunović

INPUT	
ρ (kg/m^3)	1000
Q (l/s)	26
φ (°)	90
OUTPUT	
F_x (N)	7078
F_y (N)	6385
F(N)	**9532**

Lesson 1-1
Pipe Thrust





INPUT 1-2	
L (m)	0.5
D (mm)	300
p_1 (kPa)	100
OUTPUT	
A_1 (m^2)	0.071
v_1 (m/s)	0.37
p_1A_1 (N)	7069
pQv_1 (N)	10

INPUT 2-3	
L (m)	0.5
D (mm)	300
p_3 (kPa)	100
OUTPUT	
A_3 (m^2)	0.071
v_3 (m/s)	0.37
p_3A_3 (N)	7069
Qv_3 (N)	10

w (N)	693

Lesson 1-1 Pipe Thrust

Contents:
Calculation of the pipe thrust in a pipe bend.

Goal:
Sensitivity analysis of the forces acting on the pipe bend, namely from the water pressure, flow conveyance and the water weight.

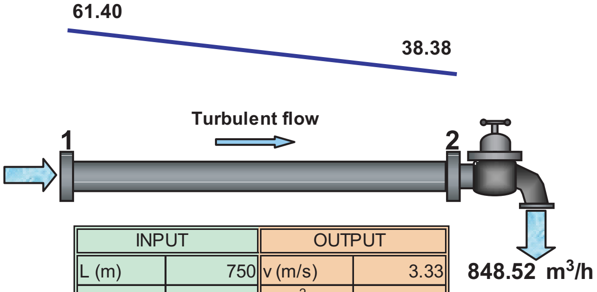
Abbreviations:

INPUT	
p (kg/m^3)	Mass density of water
Q (l/s)	Flow rate
φ (°)	Angle of bend (degrees)
OUTPUT	
F_x (N)	Thrust force in horizontal direction
F_y (N)	Thrust force in vertical direction
F (N)	Combined force
w (N)	Weight of water in the isolated section

INPUT	
L (m)	Pipe length
D (mm)	Pipe diameter
p_1 (kPa)	Pressure in cross-section 1 (3)
OUTPUT	
A_1 (m^2)	Cross-section area 1 (3)
v_1 (m/s)	Velocity in cross-section 1 (3)
p_1A_1 (N)	Force from the pressure acting on cross-section 1 (3)
pQv_1 (N)	Dynamic force acting on cross-section 1 (3)

Notes:
The calculation ultimately yields the combined force F from the balance of all forces acting in the cross-sections 1 and 3, including the weight of the water. The water weight is calculated from the pipe lengths and diameters in the sections 1-2-3. If this is to be neglected, some of these parameters should be set to 0. The thrust forces (horizontal, vertical and combined) including the force from the water weight are plotted in the graph.

Lesson 1-2
Hydraulic Grade Line



INPUT		OUTPUT	
L (m)	750	v (m/s)	3.33
D (mm)	300	u (m^2/s)	1.31E-06
k (mm)	0.1	Re (-)	765,651
Q (l/s)	235.7	λ (-)	0.0162
T (ºC)	10	**h_f (mwc)**	**23.02**
H_2 (msl)	38.38	S (-)	0.0307

Lesson 1-2 Hydraulic Grade Line

Contents:
Calculation of the friction losses in a single pipe (application of the Darcy–Weisbach formula).

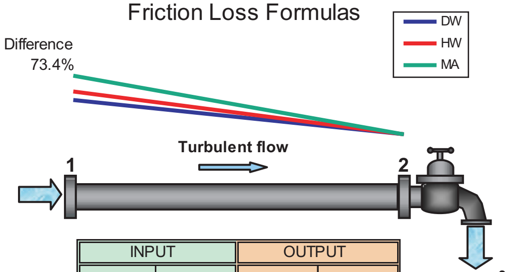
Goal:
Sensitivity analysis of the basic hydraulic parameters, namely the pipe length, diameter, internal roughness and flow rate, and water temperature.

Abbreviations:

L (m)	Pipe length	v (m/s)	Flow velocity
D (mm)	Pipe diameter	u (m^2/s)	Kinematic viscosity
k (mm)	Internal roughness	Re (-)	Reynolds number
Q (l/s)	Flow rate	λ(-)	Darcy–Weisbach friction factor
T (ºC)	Water temperature (degrees Celsius)	h_f (mwc)	Friction loss (metres of water column)
H_2 (msl)	Downstream piezometric head (metres above sea level)	S (-)	Hydraulic gradient

Notes:
The calculation ultimately yields the upstream piezometric head required to maintain the specified downstream head for the specified values of L,D,k & Q.

Lesson 1-3
Friction Loss Formulas



INPUT		OUTPUT	
D (mm)	800	v (m/s)	1.99
Q (l/s)	1000	U(m^2/s)	1.31E-06
T (ºC)	10	Re (-)	1,218,155
k (mm)	0.2	**S_{dw} (-)**	**0.0038**
C_{hw} (-)	115	**S_{hw} (-)**	**0.0048**
N($m^{-1/3}$s)	0.014	**S_{ma} (-)**	**0.0066**

3600.00 m^3/h

Lesson 1-3 Friction Loss Formulae

Contents:

Single pipe calculation of the hydraulic gradients using the Darcy–Weisbach, Hazen–Williams and Manning formulae.

Goal:

Comparison of the calculation accuracy and sensitivity of the Darcy–Weisbach, Hazen–Williams and Manning friction factors.

Abbreviations:

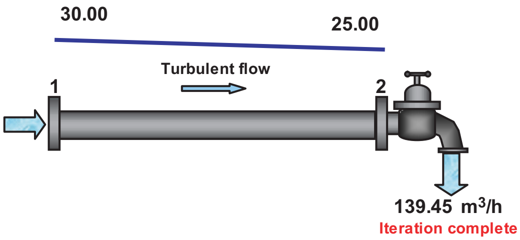
D (mm)	Pipe diameter
Q (l/s)	Flow rate
T (ºC)	Water temperature
k (mm)	Internal roughness
C_{hw} (-)	Hazen–Williams friction factor
N($m^{-1/3}$s)	Manning friction factor

v (m/s)	Flow velocity
u (m^2/s)	Kinematic viscosity
Re (-)	Reynolds number
S_{dw} (-)	Hydraulic gradient determined by the Darcy–Weisbach formula
S_{hw} (-)	Hydraulic gradient determined by the Hazen–Williams formula
S_{ma} (-)	Hydraulic gradient determined by the Manning formula

Note:

The percentage shows the difference between the lowest and the highest value of the three hydraulic gradients.

Lesson 1-4
Maximum Capacity



INPUT		OUTPUT	
L (m)	500	h_f (mwc)	5.00
D (mm)	200	U(m^2/s)	1.24E-06
k (mm)	0.5	Re (-)	199,147
S (-)	0.01	λ(-)	0.0258
T (ºC)	12	v (m/s)	1.23
H_2 (msl)	25	**Q (l/s)**	**38.74**

Assumption

v (m/s)	1.23

Lesson 1-4 Maximum Capacity

Contents:
Single pipe calculation using the Darcy–Weisbach formula.

Goal:
Determination of the maximum flow rate in a pipe of a specified diameter and hydraulic gradient.

Abbreviations:

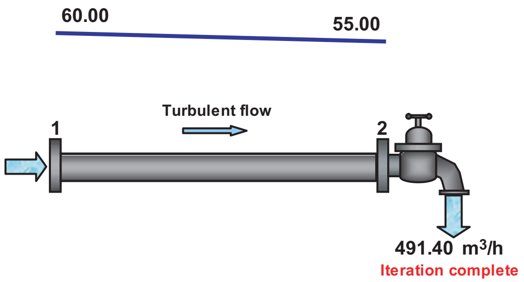
L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
S (-)	Hydraulic gradient
T (ºC)	Water temperature
H_2 (msl)	Downstream piezometric head

h_f (mwc)	Friction loss
u (m^2/s)	Kinematic viscosity
Re (-)	Reynolds number
λ(-)	Darcy–Weisbach friction factor
v (m/s)	Calculated flow velocity
Q (l/s)	Flow rate

v (m/s)	Assumed flow velocity

Notes:
The iterative procedure starts by assuming the flow velocity (commonly 1 m/s) required for determination of the Reynolds number i.e. the friction factor.
The velocity calculated afterwards by the Darcy–Weisbach formula serves as an input for the next iteration.
The iterative process is achieved by typing the value of the calculated velocity into the cell of the assumed velocity.
The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.

Lesson 1-5
Optimal Diameter



INPUT		OUTPUT	
L (m)	500	h_f (mwc)	5.00
k (mm)	0.1	u(m^2/s)	1.31E-06
Q (l/s)	136.5	**D (mm)**	**303**
S (-)	0.01	Re (-)	438,667
T (ºC)	10	λ(-)	0.0168
H_2 (msl)	55	v (m/s)	1.88

Assumption

v (m/s)	1.89

Lesson 1-5 Optimal Diameter

Contents:
Single pipe calculation using the Darcy–Weisbach formula.

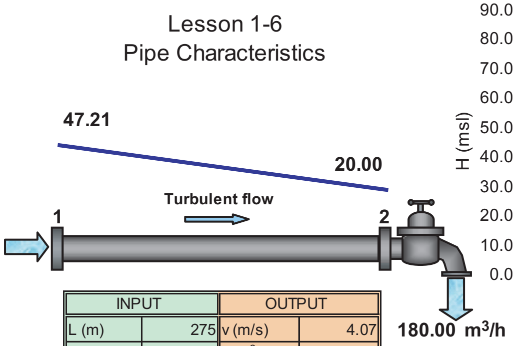
Goal:
Determination of the optimal pipe diameter for a specified flow rate and hydraulic gradient.

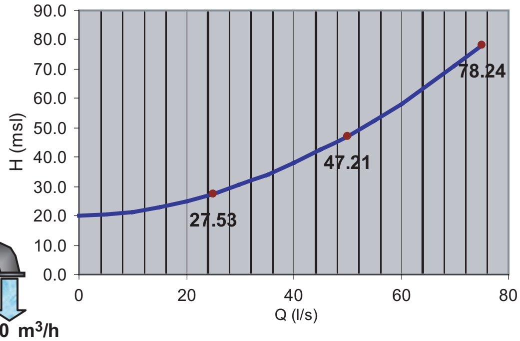
Abbreviations:

L (m)	Pipe length	h_f (mwc)	Friction loss
k (mm)	Internal roughness	u (m^2/s)	Kinematic viscosity
Q (l/s)	Flow rate	D (mm)	Pipe diameter
S (-)	Hydraulic gradient	Re (-)	Reynolds number
T (ºC)	Water temperature	λ (-)	Darcy–Weisbach friction factor
H_2 (msl)	Downstream piezometric head	v (m/s)	Calculated flow velocity

v (m/s)	Assumed flow velocity

Notes:
This is the same iterative procedure as in Lesson 1-4, except that the pipe diameter is determined from the assumed/calculated velocity (and specified flow rate).
The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.





INPUT		OUTPUT	
L (m)	275	v (m/s)	4.07
D (mm)	125	u(m^2/s)	1.24E-06
k (mm)	0.01	Re (-)	412,295
Q (l/s)	50	λ(-)	0.0146
T (°C)	12	**h_f (mwc)**	**27.21**
H_2 (msl)	20	S (-)	0.0989

Lesson 1-6 Pipe Characteristics

Contents:
Friction loss calculation in a single pipe of specified length, diameter and roughness.

Goal:
Determination of the pipe characteristics diagram.

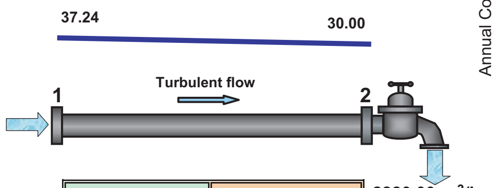
Abbreviations:

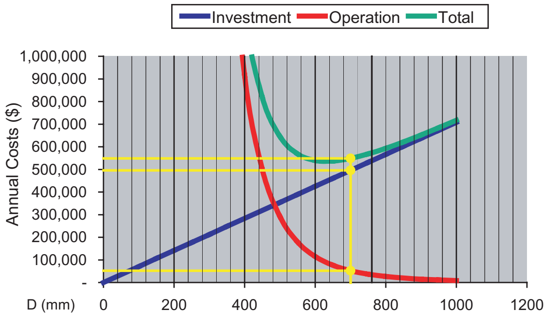
L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate
T (ºC)	Water temperature
H_2 (msl)	Downstream piezometric head

v (m/s)	Flow velocity
u (m^2/s)	Kinematic viscosity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
S (-)	Hydraulic gradient

Notes:
The friction loss is calculated for the flow range 0-1.5Q (specified) in the same way as in Lesson 1-2, and the results are plotted in the graph.
The three points selected in the graph show the upstream heads required to maintain the specified downstream head for 0.5Q, Q and 1.5Q, respectively.
It is assumed the reference level at the pipe axis makes the downstream piezometric head equal to the pressure and the static head of the pipe characteristics.
The friction loss at the same curve represents its dynamic head.

Lesson 1-7
The Most Economic
Diameter





2880.00 m^3/h

INPUT		OUTPUT	
L (m)	750	v (m/s)	2.83
D (mm)	600	u (m^2/s)	1.31E-06
k (mm)	0.1	Re (-)	1,299,366
Q (l/s)	800	λ(-)	0.0142
T (ºC)	10	h_f (mwc)	7.24
Eff. (%)	65%	P (kW)	87.39
e ($/KWh)	0.1	EC ($/y)	76,553

H_2 (msl)	30

ΔD (mm)
100

D_{opt} (mm)
606

INPUT		OUTPUT	
a (-)	13000	PW ($)	3,988,926
n (years)	30	$a_{n/r}$ (-)	0.088883
r (%)	8.00%	A ($/y)	354,326
Start (year)	2005	Total ($/y)	430,879

Phased investment

Phases	1	2	3
In year	2005	2105	2025
Invest.(%)	50%	30%	20%

Lesson 1-7 The Most Economic Diameter

Contents:

Calculation of annual investment and operational costs for a single pipe, based on the present worth method.

Goal:

To select the most economic pipe diameter for the given flow rate and specified conditions of the bank loan. The analysis includes a possibility of phased investment.

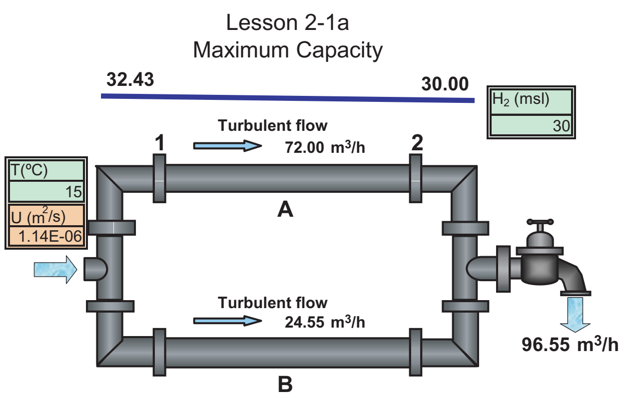
Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate
T (ºC)	Water temperature (degrees Celsius)
Eff. (%)	Assumed pump efficiency at flow Q
e ($/KWh)	Energy tariff
H_2 (msl)	Downstream piezometric head (metres above sea level)
ΔD (mm)	Diameter increment (X-axis on the diagram)
a (-)	Pipe cost factor (Cost = aD in $ for D expressed in metres)
n (years)	Loan repayment period
r (%)	Interest rate
Start (year)	Year in which the repayment starts (blank or 0 means 'immediately')
In year	Year of the phased investment (blank or 0 means 'no phased investment')
Invest.(%)	Percentage of the total investment in particular phase (blank or 0 means 'no phased investment')

v (m/s)	Flow velocity
u (m^2/s)	Kinematic viscosity
Re (-)	Reynolds number
λ(-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss (metres of water column)
P (kW)	Power consumption at flow Q
EC ($/y)	Annual energy (operational) costs
D_{opt} (mm)	Optimal diameter (Equation 4.9, Section 4.1.2 in the book)
PW ($)	Present worth of the total investment (at the start of repayment)
$a_{n/r}$ (-)	Annuity
A ($/y)	Annual instalment for the loan repayment (investment cost)
Total ($/y)	Total annual investment and operational costs

Notes:

The simplified procedure is based on the assumption that all input parameters remain constant throughout the loan repayment period (average values).
In addition, the effect of inflation has been neglected. The loan repayment can start immediately or be delayed for a selected number of years,
which increases the annual instalments. On the other hand, phased investments can reduce these instalments.
The optimum diameter is calculated based on the fixed friction factor, the result serves as an indication for selection of the input diameter.
The diagram with the investment and operational costs has been plotted in 20 points for the range of diameters between 0 and 10ΔD. The range of the Y-axis
has been fixed between 0 and 1,000,000 $. In the case of larger pipes, this range may need to be widened and therefore the worksheet protection has to be
temporarily removed ('Excel' menu command **Tools>>Protection>>Unprotect Sheet**).



INPUT - A		OUTPUT - A	
L (m)	275	v (m/s)	1.13
D (mm)	150	Re (-)	148,934
k (mm)	0.1	λ(-)	0.0203
Q (l/s)	20	**h_f (mwc)**	**2.43**
		S (-)	0.0088

Maximum Capacity

INPUT - B		OUTPUT - B	
L (m)	275	Re (-)	76,325
D (mm)	100	λ(-)	0.0230
k (mm)	0.1	v (m/s)	0.87
v (m/s)	0.87	**Q (l/s)**	**6.82**
		S (-)	0.0088

Iteration complete

$$hf = hf_A = hf_B \quad ; \quad Q = Q_A + Q_B$$

Lesson 2-1a Pipes in Parallel – Maximum Capacity

Contents:
Hydraulic calculation of two pipes connected in parallel.

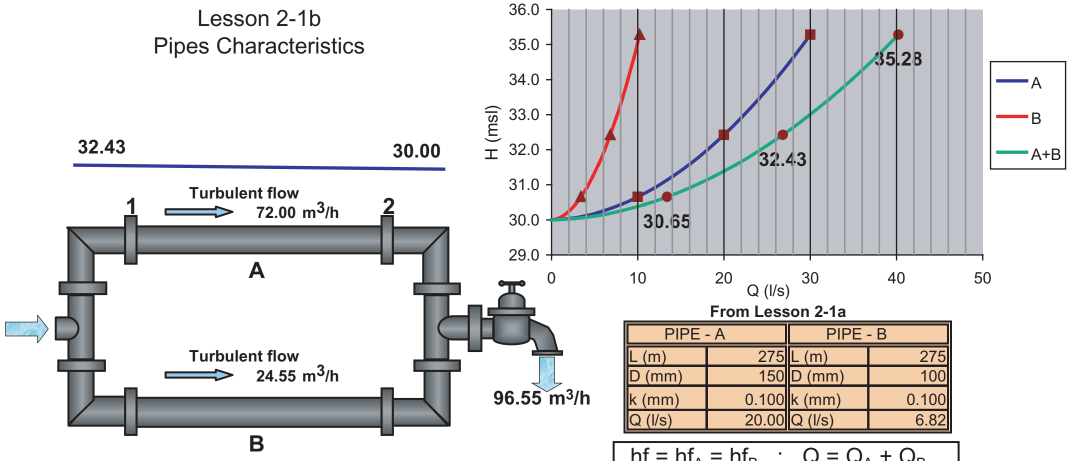
Goal:
Resulting from the demand growth, a new pipe (B) of a specified diameter is to be laid in parallel, next to the existing one (A). The task is to find the maximum flow rate in this pipe by maintaining the same friction loss as in the existing pipe.

Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate in the existing pipe
T (ºC)	Water temperature
H_2 (msl)	Downstream piezometric head
v (m/s)	Assumed flow velocity in the new pipe

h_f (mwc)	Friction loss
u (m^2/s)	Kinematic viscosity
Re (-)	Reynolds number
λ(-)	Darcy–Weisbach friction factor
v (m/s)	Calculated flow velocity
Q (l/s)	Flow rate in the new pipe
S (-)	Hydraulic gradient

Notes:
The friction loss in the existing pipe is calculated as in Lesson 1-2. Its hydraulic gradient is used as an input for calculation of the maximum capacity in the new pipe.
The same iterative procedure as in Lesson 1-4 applies for the new pipe.
The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.



From Lesson 2-1a

PIPE - A		PIPE - B	
L (m)	275	L (m)	275
D (mm)	150	D (mm)	100
k (mm)	0.100	k (mm)	0.100
Q (l/s)	20.00	Q (l/s)	6.82

$hf = hf_A = hf_B \quad ; \quad Q = Q_A + Q_B$

Lesson 2-1b Pipes in Parallel – Pipe Characteristics

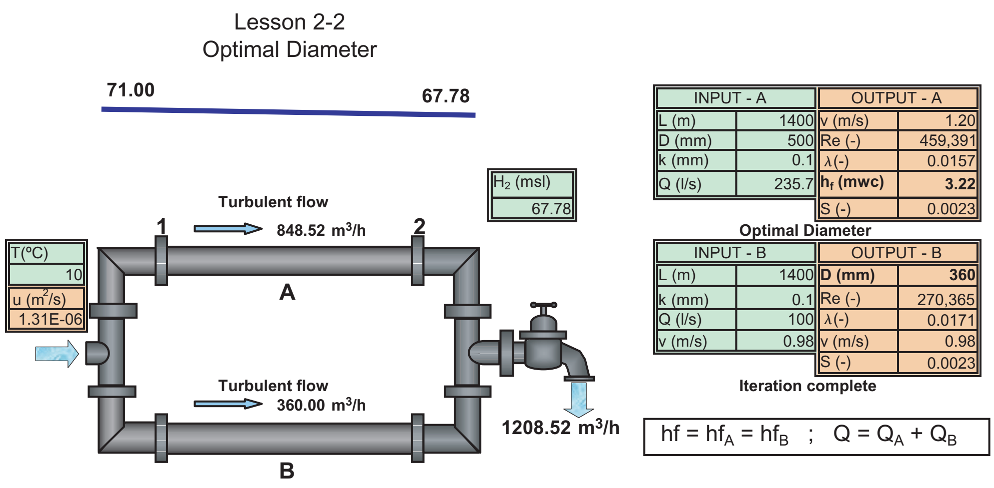
Contents:
Hydraulic calculation of two pipes connected in parallel.

Goal:
Determination of the pipe characteristics diagrams for the system from Lesson 2-1a.

Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate in the existing pipe

Notes:
The pipe characteristics diagram is presented for each of the pipes and both of them operating in parallel, in the range 0-1.5Q (=$Q_A + Q_B$).
The three points selected in the graph show the upstream head required to maintain the specified downstream head for 0.5Q, Q and 1.5Q, respectively.
The flow rate in each of the pipes can be determined from this diagram.



Lesson 2-2 Pipes in Parallel – Optimal Diameter

Contents:

Hydraulic calculation of two pipes connected in parallel.

Goal:

Resulting from the demand growth, a new pipe (B) is to be laid in parallel, next to the existing one (A).
The task is to determine the optimal diameter of this pipe for a given flow rate, by maintaining the same friction loss as in the existing pipe.

Abbreviations:

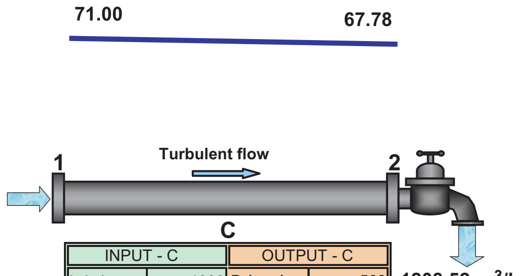
L (m)	Pipe length
D (mm)	Diameter of the existing pipe
k (mm)	Internal roughness
Q (l/s)	Flow rate
T (ºC)	Water temperature
H_2 (msl)	Downstream piezometric head
v (m/s)	Assumed flow velocity in the new pipe

h_f (mwc)	Friction loss
u (m^2/s)	Kinematic viscosity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
v (m/s)	Calculated flow velocity
D (mm)	Diameter of the new pipe
S (-)	Hydraulic gradient

Notes:

The friction loss in the existing pipe is calculated as in Lesson 1-2. Its hydraulic gradient is used as an input for calculation of the maximum capacity in the new pipe.
The same iterative procedure as in Lesson 1-5 applies for the new pipe.
The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.

Lesson 2-3
Equivalent Diameter



From Lesson 2-2

T (ºC)	10	U (m²/s)	1.31E-06
H_2 (msl)	67.78	h_f (mwc)	3.22

PIPE - A		PIPE - B	
L (m)	1400	L (m)	1400
D (mm)	500	D (mm)	360
k (mm)	0.100	k (mm)	0.100
v (m/s)	1.20	v (m/s)	0.98
Q (l/s)	235.70	Q (l/s)	100.00

INPUT - C		OUTPUT - C	
L (m)	1000	**D (mm)**	**563**
k (mm)	0.1	Re (-)	581,407
v (m/s)	1.35	λ(-)	0.0151
Q (l/s)	335.70	v (m/s)	1.53

Iterate the velocity (diameter)!

$$hf_C = hf_A = hf_B \quad ; \quad Q_C = Q_A + Q_B$$

Lesson 2-3 Pipes in Parallel – Equivalent Diameter

Contents:

Calculation of a hydraulically equivalent pipe.

Goal:

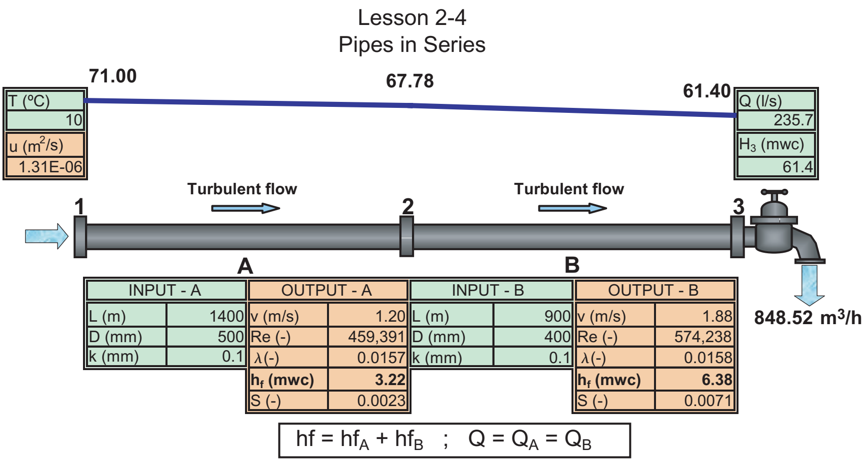
As an alternative to the system in Lesson 2-2, one larger pipe can be laid instead of two parallel pipes.
The task is to determine the optimal diameter of this pipe for a given flow rate, by maintaining the same hydraulic gradient as in the existing pipe.

Abbreviations:

The same as in Lesson 2-2.

Notes:

The total flow rate and hydraulic gradient from Lesson 2-2 are used as an input for calculation of the optimal pipe diameter.
The same iterative procedure as in Lesson 1-5 applies.
The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.



Lesson 2-4 Pipes in Series – Hydraulic Grade Line

Contents:

Calculation of the friction losses in two pipes connected in series.

Goal:

Resulting from the system expansion, a new pipe (B) of a specified diameter is to be laid in series, after the existing one (A).
The task is to determine the piezometric head at the upstream side (H_1) required to maintain the minimum head at the downstream side (H_3).

Abbreviations:

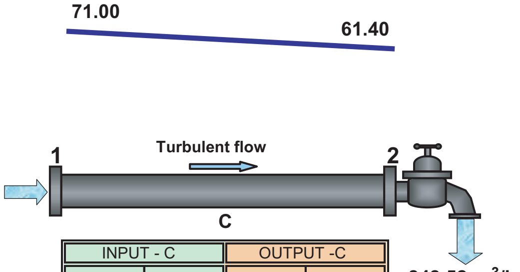
L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate
H_3 (msl)	Downstream piezometric head
T (ºC)	Water temperature

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ(-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
S (-)	Hydraulic gradient
u (m^2/s)	Kinematic viscosity

Notes:

By maintaining the same flow rate in pipes A and B, the same calculation procedure applies as in Lesson 1-2.

Lesson 2-5
Equivalent Diameter



INPUT - C		OUTPUT -C	
L (m)	550	**D (mm)**	**359**
k (mm)	0.01	Re (-)	640,023
v (m/s)	2.33	λ(-)	0.0131
h_f (mwc)	9.60	v (m/s)	3.07

Iterate the velocity(diameter)!

From Lesson 2-4

T (ºC)	10	U (m²/s)	1.31E-06
H_3 (msl)	61.4	Q (l/s)	235.70

PIPE - A		PIPE - B	
L (m)	1400	L (m)	900
D (mm)	500	D (mm)	400
k (mm)	0.100	k (mm)	0.100
v (m/s)	1.20	v (m/s)	1.88
h_f (mwc)	3.22	h_f (mwc)	6.38

$$hf_C = hf_A = hf_B \quad ; \quad Q_C = Q_A + Q_B$$

Lesson 2-5 Pipes in Series – Equivalent Diameter

Contents:
Calculation of a hydraulically equivalent pipe.

Goal:
As an alternative to the system in Lesson 2-4, one longer pipe is to be laid instead of the two serial pipes.
The task is to determine the optimal diameter of this pipe for a given flow rate, by maintaining the existing head difference between the points 1 and 3.

Abbreviations:
The same as in Lesson 2-4.

Notes:
The flow rate and piezometric head difference (H_1-H_3 i.e. hf_A+hf_B) from Lesson 2-4 are used as an input for calculation of the optimal pipe diameter.
The same iterative procedure applies as in Lesson 1-5.
The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.

Lesson 3-1
Residual
Pressures

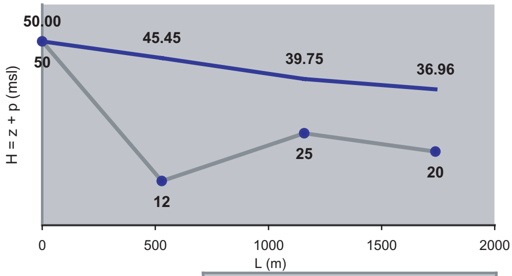
Q_n total	T (ºC)	U (m^2/s)
75.6	10	1.3E-06

PATH
p1
p3
p5

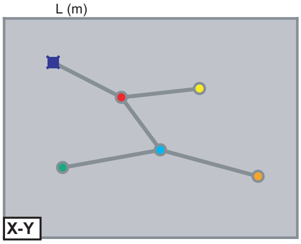
ups → dws

NODES	X (m)	Y (m)	Z (msl)	Q_n (l/s)	H (msl)	p (mwc)
n1	25	200	50	0	50	0.00
n2	60	160	12	10.4	45.45	33.45
n3	100	170	22	22.1	41.02	19.02
n4	30	80	17	10.2	29.37	12.37
n5	80	100	25	18.5	39.75	14.75
n6	130	70	20	14.4	36.96	16.96

k (mm)
0.1



PIPES	N_{ups}	N_{dws}	L_{xy}(m)	L (m)	D (mm)	Q (l/s)	v (m/s)	Re (-)	λ (-)	S (-)	h_f (mwc)
p1	n1	n2	53	530	250	75.60	1.54	294,696	0.0178	0.0086	4.55
p2	n2	n3	41	410	150	22.10	1.25	143,580	0.0203	0.0108	4.43
p3	n2	n5	63	630	200	43.10	1.37	210,010	0.0189	0.0090	5.70
p4	n5	n4	54	540	100	10.20	1.30	99401	0.0224	0.0192	10.38
p5	n5	n6	58	580	150	14.40	0.81	93554	0.0213	0.0048	2.78



Lesson 3-1 Branched Network Layouts – Residual Pressures

Contents:

Calculation of the friction losses in a branched network configuration.

Goal:

Pressures in the system should be determined for a specified network configuration, distribution of nodal demands and piezometric head fixed in the source node (n1).

Abbreviations:

NODES	Node data
X (m)	Horizontal co-ordinate
Y (m)	Vertical co-ordinate
Z (msl)	Altitude
Q_n (l/s)	Nodal demand
H (msl)	Piezometric head
p (mwc)	Nodal pressure
Q_n total	Total demand of the system
T (ºC)	Water temperature
u (m^2/s)	Kinematic viscosity
PATH	Pipes selected to be plotted with their piezometric heads. (starting from the upstream- to the downstream pipes)
k (mm)	Internal roughness (uniform for all pipes)

PIPES	Pipe data
N_{ups}	Upstream node
N_{dws}	Downstream node
L_{xy} (m)	Length calculated from the X/Y co-ordinates
L (m)	Length adopted for the hydraulic calculation
D (mm)	Diameter
Q (l/s)	Flow rate
v (m/s)	Flow velocity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
S (-)	Hydraulic gradient
h_f (mwc)	Friction loss

Notes:

The nodes are plotted based on the X/Y input (origin of the graph is at the lower left corner). Any node name can be used.
The first node in the list of nodes simulates the source and therefore has a fixed piezometric head.
The pipes are plotted based on the N_{ups}/N_{dws} input. This input determines connectivity between the nodes and hence the flow rates/directions.
From the determined pipe flows, the friction losses and consequently the nodal heads/pressures will be calculated.
Each node may appear only once as a downstream node (N_{dws}). Doing otherwise suggests a system consisting of more than one source, or from loops.

Lesson 3-2
Optimal
Diameters

Q_n total	T (ºC)	u (m²/s)
131	10	1.3E-06

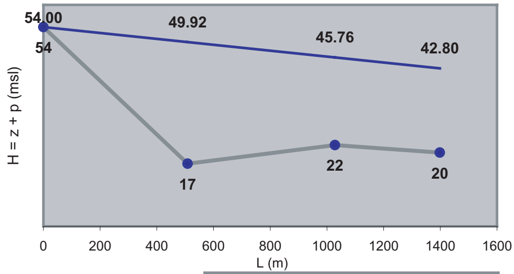
PATH
p3
p5
p6

ups ↓ dws

NODES	X (m)	Y (m)	Z (msl)	Q_n (l/s)	H (msl)	p (mwc)
n1	50	135	54	0	54	0.00
n2	25	200	12	10.4	48.56	36.56
n3	60	46	22	18.5	46.56	24.56
n4	65	88	17	12.2	49.92	32.92
n5	100	100	25	22.1	45.12	20.12
n6	130	20	20	14.4	42.80	22.80
n7	81	39	22	13.8	45.76	23.76
n8	99	154	38	12.5	49.44	11.44
n9	60	180	33	19.1	45.36	12.36
n10	12	134	28	8	43.20	15.20

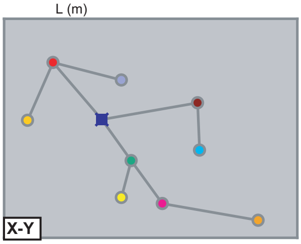
S (-)
0.008

k (mm)
0.1



Iteration complete

PIPES	N_{ups}	N_{dws}	L_{xy}(m)	L (m)	Q (l/s)	v (m/s)	D (mm)	Re (-)	λ(-)	v (m/s)	h_f (mwc)	D (mm)
p1	n1	n8	53	570	34.60	1.24	189	178,786	0.0193	1.24	4.56	200
p2	n8	n5	54	540	22.10	1.11	159	135,267	0.0203	1.11	4.32	200
p3	n1	n4	49	510	58.90	1.41	231	248,841	0.0182	1.41	4.08	250
p4	n4	n3	42	420	18.50	1.06	149	121,088	0.0207	1.06	3.36	150
p5	n4	n7	52	520	28.20	1.18	175	157,426	0.0197	1.18	4.16	200
p6	n7	n6	53	370	14.40	1.00	135	103,586	0.0213	1.00	2.96	150
p7	n1	n2	70	680	37.50	1.26	194	187,962	0.0191	1.26	5.44	200
p8	n2	n10	67	670	8.00	0.86	109	71,786	0.0229	0.86	5.36	150
p9	n2	n9	40	400	19.10	1.07	151	123,519	0.0206	1.07	3.20	150



Lesson 3-2 Branched Network Layouts – Optimal Diameters

Contents:
Hydraulic calculation of a branched network configuration.

Goal:
The pipe diameters in the system should be determined for a specified network configuration, distribution of nodal demands and uniform (= design) hydraulic gradient.

Abbreviations:

NODES	Node data
X (m)	Horizontal co-ordinate
Y (m)	Vertical co-ordinate
Z (msl)	Altitude
Q_n (l/s)	Nodal demand
H (msl)	Piezometric head
p (mwc)	Nodal pressure

Q_n total	Total demand of the system
T (ºC)	Water temperature
u (m²/s)	Kinematic viscosity

PATH	Pipes selected to be plotted with their piezometric heads. (starting from the upstream to the downstream pipes)
S (-)	Design hydraulic gradient (uniform for all pipes)
k (mm)	Internal roughness (uniform for all pipes)

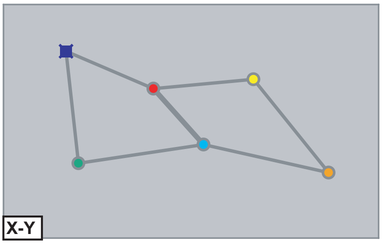
PIPES	Pipe data
N_{ups}	Upstream node
N_{dws}	Downstream node
L_{xy} (m)	Length calculated from the X/Y co-ordinates
L (m)	Length adopted for hydraulic calculation
Q (l/s)	Flow rate
v (m/s)	Flow velocity of iteration 'I'
D (mm)	Calculated diameter
Re (-)	Reynolds number
λ(-)	Darcy–Weisbach friction factor
v (m/s)	Flow velocity of iteration 'i+1'
h_f (mwc)	Friction loss
D (mm)	Adopted diameter (manufactured size)

Notes:
The procedure of the network building is the same as in Lesson 3-1. The order of the nodes from upstream to downstream has to be respected in the list of pipes.
The first node in the list of nodes simulates the source and therefore has a fixed piezometric head.
The hydraulic calculation follows the principles of the single pipe calculation from Lesson 1-5; the iteration procedure has to be conducted for all pipes.
This can be done at once, by copying the entire column of the 'i+1' velocities, and pasting it subsequently to the column of 'I' velocities.
Only the 'Excel' command **Edit>>Paste Special>>Values** should be used in this case; the ordinary 'Paste' command also copies the cell formulae, which is wrong!
The message **Iteration complete** appears once the total difference between the velocities in two iterations drops below 0.01 m/s.

Lesson 4-1
Method of
Balancing Heads

Q_n total	T (°C)	U (m²/s)
75.6	10	1.3E-06

NODES	X (m)	Y (m)	Z (msl)	Q_n (l/s)	H (msl)	p (mwc)
n1	25	200	50	0	50	0.00
n2	60	160	12	10.4	47.88	35.88
n3	100	170	22	22.1	44.22	22.22
n4	30	80	17	10.2	67.70	50.70
n5	80	100	25	18.5	46.53	21.53
n6	130	70	20	14.4	45.28	25.28



k (mm)	0.1

PIPES	$N1_{cw}$	$N2_{cw}$	L_{xy}(m)	L (m)	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)	Q (l/s)
p1	n1	n2	53	530	250	50.60	1.03	2.12	61.56
p2	n2	n5	63	630	200	20.20	0.64	1.36	29.95
p3	n5	n4	54	540	100	-14.80	-1.88	-21.17	-3.84
p4	n4	n1	120	1200	100	-25.00	-3.18	-129.89	-14.04
						0.00			0.00
LOOP 1					δQ (l/s)=	10.96	Sum=	-147.59	

Iterate Q!

PIPES	$N1_{cw}$	$N2_{cw}$	L_{xy}(m)	L (m)	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)	Q (l/s)
p2	n5	n2	63	630	200	-20.20	-0.64	-1.36	-29.95
p5	n2	n3	41	410	150	20.00	1.13	3.66	21.21
p6	n3	n6	104	1040	100	-2.10	-0.27	-1.06	-0.89
p7	n6	n5	58	580	150	-16.50	-0.93	-3.60	-15.29
						0.00			0.00
LOOP 2					δQ (l/s)=	1.21	Sum=	-2.35	

Iterate Q!

PIPES	$N1_{cw}$	$N2_{cw}$	L_{xy}(m)	L (m)	D (mm)	Q (l/s)	v (m/s)	h_f (mwc)	Q (l/s)
									0.00
									0.00
									0.00
									0.00
									0.00
LOOP 3					δQ (l/s)=	0.00	Sum=	0.00	

Iteration complete

Lesson 4-1 Looped Network Layouts – Method of Balancing Heads

Contents:

Hydraulic calculation of a looped network configuration by the Hardy-Cross Method of Balancing Heads (Loop Oriented Method).

Goal:

The flows and pressures in the system should be determined for a specified network configuration, nodal demands and piezometric head fixed in a source node.

Abbreviations:

NODES	Node data
X (m)	Horizontal co-ordinate
Y (m)	Vertical co-ordinate
Z (msl)	Altitude
Q_n (l/s)	Nodal demand
H (msl)	Piezometric head
p (mwc)	Nodal pressure
Q_n total	Total demand of the system (l/s)
T (°C)	Water temperature
U (m²/s)	Kinematic viscosity
k (mm)	Internal roughness (uniform)

PIPES	Pipe data per loop
$N1_{cw}$	Pipe node nr.1 (clockwise direction)
$N2_{cw}$	Pipe node nr.2 (clockwise direction)
L_{xy} (m)	Length calculated from the X/Y co-ordinates
L (m)	Length adopted for hydraulic calculation
D (mm)	Diameter
Q (l/s)	Flow rate of iteration 'I'
v (m/s)	Flow velocity
h_f (mwc)	Friction loss
Q (l/s)	Flow rate of iteration 'i+1'
δQ (l/s)	Flow rate correction. $Q_{i+1} = Q_i + dQ$
Sum	Sum of friction losses in the loop (clockwise direction)

Notes:

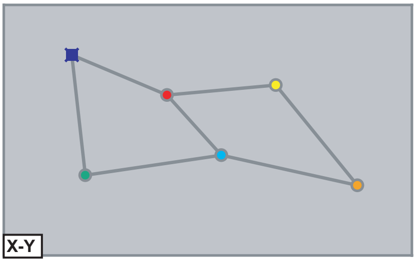
The table with the nodal data is prepared in the same way as in Lessons 3-1 and 3-2.
The pipes are plotted based on the $N1_{cw}$/$N2_{cw}$ input. As a convention, this input has to be made in a clockwise direction for each loop.
The pipes shared by neighbouring loops should appear in both tables (with opposite flow directions).
The first node in the list of nodes and pipes (in loop 1) simulates the source and therefore has a fixed piezometric head.
To provide a correct spreadsheet calculation of nodal piezometric heads, the tables of loops 2 and 3 should start with a previously filled (shared) pipe.
The iterative process starts by distributing the pipe flows 'I' arbitrarily, but satisfying the Continuity Equation in each node.
Negative flows, velocities and friction losses indicate anti-clockwise flow direction.
The flow correction (δQ) is calculated from the friction losses/piezometric heads, and flows for iteration 'i+1' are determined for all loops simultaneously.
Both δQ corrections of the neighbouring loops are applied in case of the shared pipes (with opposite signs!).
The iteration proceeds by copying the entire column of the 'i+1' flows, and pasting it subsequently to the column of "i" flows.
Only the 'Excel' command **Edit>>Paste Special>>Values** should be used in this case; the ordinary 'Paste' command also copies the cell formulae, which is wrong!
The message **Iteration complete** appears once the sum of friction losses in the loop drops below 0.01 mwc.

Lesson 4-2
Method of
Balancing Flows

Q_n total	T (ºC)	u(m^2/s)
75.6	10	1.3E-06

δH total
14.19

Iterate H!



NODES	X (m)	Y (m)	Z (msl)	Q_n (l/s)	H (msl)	p (mwc)	Q (l/s)	H (msl)	H (msl)
n1	25	200	50	0	50	0.00	-55.92	0.00	50.00
n2	60	160	12	10.4	48.00	36.00	-19.78	-1.04	46.96
n3	100	170	22	22.1	46.00	24.00	-13.85	-3.43	42.57
n4	30	80	17	10.2	42.00	25.00	-8.68	-5.85	36.15
n5	80	100	25	18.5	40.00	15.00	18.67	2.70	42.70
n6	130	70	20	14.4	38.00	18.00	3.95	1.16	39.16

Iterate v!

PIPES	N1	N2	L_{xy}(m)	L (m)	D (mm)	h_f (mwc)	S (-)	v (m/s)	Re (-)	λ (-)	v (m/s)	Q (l/s)	Q/h_f
p1	n1	n2	53	530	250	2.00	0.0038	1.44	275,127	0.0179	1.02	49.96	24.98
p2	n2	n5	63	630	200	8.00	0.0127	0.21	31,496	0.0245	1.42	44.76	5.60
p3	n2	n3	41	410	150	2.00	0.0049	0.88	100,560	0.0211	0.83	14.58	7.29
p4	n3	n6	104	1040	100	8.00	0.0077	0.88	67,443	0.0233	0.81	6.32	0.79
p5	n4	n5	54	540	100	2.00	0.0037	1.10	84,241	0.0227	0.57	4.44	2.22
p6	n5	n6	58	580	150	2.00	0.0034	0.64	73,812	0.0219	0.68	12.03	6.01
p7	n1	n4	120	1200	100	8.00	0.0067	1.11	85,335	0.0227	0.76	5.96	0.75
								0.00			0.00		
								0.00			0.00		
								0.00			0.00		

k (mm)
0.1

Lesson 4-2 Looped Network Layouts – Method of Balancing Flows

Contents:

Hydraulic calculation of a looped network configuration by the Method of Balancing Flows (Node Oriented Method).

Goal:

The flows and pressures in the system should be determined for a specified network configuration, nodal demands and piezometric head fixed in a source node.

Abbreviations:

NODES	Node data
X (m)	Horizontal co-ordinate
Y (m)	Vertical co-ordinate
Z (msl)	Altitude
Q_n (l/s)	Nodal demand
H (msl)	Piezometric head of iteration 'I'
p (mwc)	Nodal pressure
δQ (l/s)	Balance of the flow continuity equation
δH (msl)	Piezometric head correction. $H_{i+1} = H_i + \delta H$
H (msl)	Piezometric head of iteration 'i+1'

Q_n total	Total demand of the system
T (ºC)	Water temperature
u (m^2/s)	Kinematic viscosity

δH total	Sum of all δH-corrections

PIPES	Pipe data
N1	Pipe node nr.1
N2	Pipe node nr.2
L_{xy} (m)	Length calculated from the X/Y co-ordinates
L (m)	Length adopted for hydraulic calculation
D (mm)	Diameter
h_f (mwc)	Friction loss
S (-)	Hydraulic gradient
v (m/s)	Flow velocity of iteration 'I'
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
v (m/s)	Flow velocity of iteration 'i+1'
Q (l/s)	Flow rate
Q/h_f	Ratio used for calculation of δH-corrections

k (mm)	Internal roughness (uniform for all pipes)

Notes:

The table with the nodal data is prepared in the same way as in Lesson 4-1
The pipes are plotted based on the N1/N2 input. Unlike in Lesson 4-1, the order of nodes/pipes is not crucial in this case.
The first node in the list of nodes simulates the source and has therefore fixed piezometric head.
The heads in other nodes are distributed arbitrarily in the first iteration, except that no nodes should be allocated the same value.
The calculation starts by iterating the velocities in order to determine the pipe flows for given piezometric heads. This is done in the same way as in Lesson 3-2.
The message **Iteration complete** appears once the total difference between the velocities in two iterations drops below 0.01 m/s.
After the pipe flows have been determined, the correction (δH) is calculated and the iteration of piezometric heads proceeds.
A consecutive iteration is done node by node, by typing the current 'H_{i+1}' value into 'H_i' cell. Copying the entire column does not lead to a convergence.
The new values of the nodal piezometric heads should result in a gradual reduction of the 'δH total' value; the velocities (flows) have to be re-iterated.
The message **Iteration complete** appears once the sum of δH-corrections for all nodes drops below 0.01 mwc.

Lesson 4-3
Linear Theory

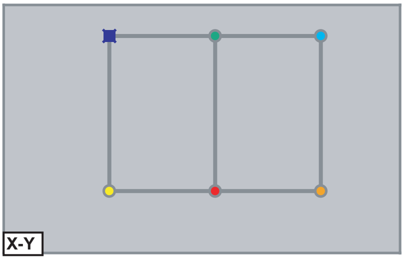
Q_n total	T (ºC)	u (m^2/s)	δH total
72.9	10	1.3E-06	0.06

Iteration complete

NODES	X (m)	Y (m)	Z (msl)	Q_n (l/s)	H (msl)	p (mwc)	Q (l/s)	H (msl)
n1	40	70	54	0	60	6.00	-72.59	60.00
n2	80	20	12	10.4	55.88	55.87	-0.25	55.87
n3	40	20	17	12.2	58.07	41.07	-0.11	58.06
n4	80	70	25	22.1	56.06	31.06	-0.12	56.06
n5	120	70	20	14.4	52.64	32.64	0.04	52.65
n6	120	20	22	13.8	50.04	28.04	0.13	50.08

(-)
1
k (mm)
0.1

X-Y



Iteration complete

PIPES	N1	N2	L_{xy}(m)	L (m)	D (mm)	Q(l/s)	v (m/s)	Re (-)	λ(-)	1/U	H1/U	H2/U	h_f (mwc)	S (-)	v (m/s)	Q (l/s)
p1	n1	n4	40	400	200	45.05	1.43	219,531	0.0188	0.0114	0.69	0.64	3.94	0.0098	1.43	45.06
p2	n4	n2	50	500	150	3.54	0.20	23,017	0.0265	0.0196	1.10	1.09	0.18	0.0004	0.20	3.53
p3	n2	n3	40	400	150	-15.44	-0.87	100,333	0.0211	0.0071	0.39	0.41	-2.19	0.0055	0.87	-15.44
p4	n3	n1	50	500	200	-27.54	-0.88	134,170	0.0197	0.0143	0.83	0.86	-1.93	0.0039	0.88	-27.54
p5	n4	n5	40	400	150	19.54	1.11	126,952	0.0206	0.0057	0.32	0.30	3.42	0.0086	1.11	19.54
p6	n5	n6	50	500	100	5.11	0.65	49,790	0.0242	0.0020	0.10	0.10	2.60	0.0052	0.65	5.10
p7	n6	n2	40	400	100	-8.83	-1.12	86,071	0.0227	0.0015	0.08	0.08	-5.84	0.0146	1.12	-8.83
																0.00
																0.00
																0.00

Lesson 4-3 Looped Network Layouts – Linear Theory

Contents:

Hydraulic calculation of a looped network configuration based on the linear theory (solution by the Newton-Raphson/successive over-relaxation method).

Goal:

For a specified network configuration, nodal demands and piezometric head fixed in a source node, the flows and pressures in the system should be determined.

Abbreviations:

NODES	Node data
X (m)	Horizontal co-ordinate
Y (m)	Vertical co-ordinate
Z (msl)	Altitude
Q_n (l/s)	Nodal demand
H (msl)	Piezometric head of iteration 'i'
p (mwc)	Nodal pressure
δQ (l/s)	Balance of the flow continuity equation
H (msl)	Piezometric head of iteration 'i+1'
Q_n total	Total demand of the system
T (ºC)	Water temperature
u (m^2/s)	Kinematic viscosity
δH total	Total error between two iterations (δH = ABS(H_{i+1} - H_i))
ω (-)	Successive over-relaxation factor (value range 1.0-2.0)

PIPES	Pipe data
N1	Pipe node nr.1 name
N2	Pipe node nr.2 name
L_{xy} (m)	Length calculated from the X/Y co-ordinates
L (m)	Length adopted for hydraulic calculation
D (mm)	Diameter
Q(l/s)	Flow rate of iteration 'I'
v (m/s)	Flow velocity of iteration 'I'
? (-)	Reynolds number
1/U	Linearisation coefficient
H1/U	Ratio used for calculation of 'H_{i+1}' (from N1)
H2/U	Ratio used for calculation of 'H_{i+1}' (from N2)
h_f (mwc)	Friction loss
S (-)	Hydraulic gradient
v (m/s)	Flow velocity of iteration 'i+1'
Q (l/s)	Flow rate of iteration 'i+1'
k (mm)	Internal roughness (uniform)

Remarks:

The table with the nodal and pipe data is prepared in the same way as in Lesson 4-2.
The first node in the list of nodes simulates the source and has therefore a fixed piezometric head.
The heads in other nodes are distributed arbitrarily in the 1st iteration, except that <u>no nodes should be allocated the same value.</u>
The pipe flows in the 1st iteration are also distributed arbitrarily (commonly to fit the velocities around 1 m/s).
The calculation starts by iterating piezometric heads in the nodes, in order to determine the pipe flows in the next iteration.
A consecutive iteration is done node by node, by typing the current 'H_{i+1}' value into 'H_i' cell.
Alternative approach, by copying the entire column ('Excel' command . (**Edit >> Paste Special 'Values'**) is likely to yield a slower convergence.
The new values of nodal piezometric heads should result in a gradual reduction of the 'dH total' value; the velocities (flows) have to be re-iterated.
That is done by copying the entire 'Q_{i+1}' column into 'Q_i' cells ('Excel' command. (**Edit >> Paste Special 'Values'**).
Messages **Iteration complete** appear once the total difference between the heads (flows) in two iterations drops below 0.1 mwc (l/s).

UNESCO-IHE
Institute for Water Education

Version 1.4
October 2005

Spreadsheet Hydraulic Lessons in

Water Transport & Distribution Part II

Contents:

Scroll-down

Disclaimer

Introduction

The spreadsheet hydraulic lessons have been developed as an aid for steady state hydraulic calculations of simple Urban water and distribution networks. These are to be carried out while solving the workshop problems that should normally be calculated manually; the spreadsheet serves here as a fast check of the results. Moreover, the spreadsheet lessons will help teachers to demonstrate a wider range of problems in a clear way, as well as to allow the students to continue analysing them at home. Ultimately, a real understanding of the hydraulic concepts will be reached through 'playing' with the data.

Over forty problems have been classified in eight groups/worksheets according to the contents of the book 'Introduction to Urban Water Distribution' by N. Trifunović. This book covers a core curriculum of 'Water Transport and Distribution 1', which is a three-week module in theWater Supply Engineering specialisation at UNESCO-IHE. Outside the regular MSc programme, this module is also offered as a stand-alone short course/distance learning package.

To be able to use the spreadsheet, brief accompanying instructions for each exercise are given in the 'About' worksheet (see below).
Each layout covers approximately one full screen (30 rows) consisting of drawings, tables and graphs. In the tables:

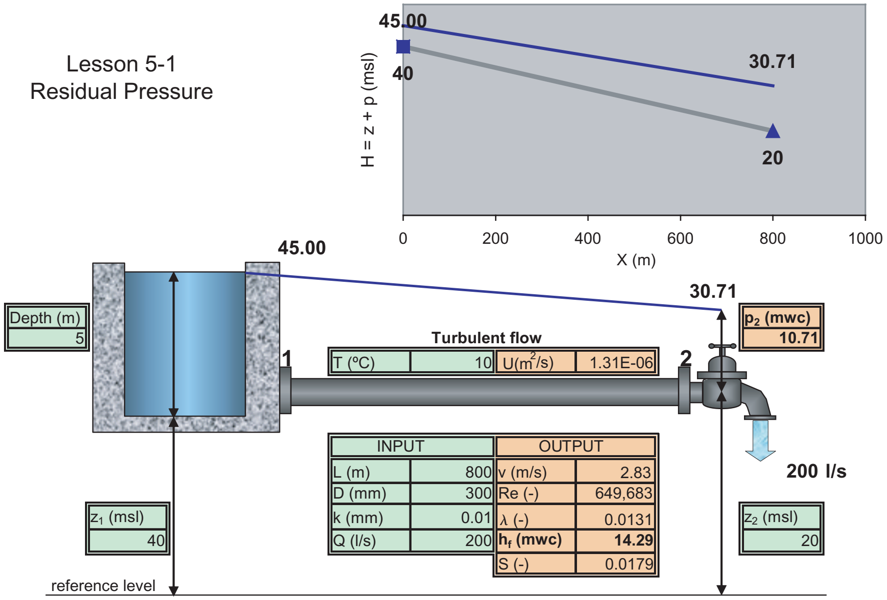
The green colour indicates input cells. Except for the headers, these cells are unprotected and their contents are used for calculations.
The brown colour indicates output cells. These cells contain fixed formulae and are therefore protected.

Moreover, some intermediate calculations have been moved further to the right in the worksheet, being irrelevant for educational purposes.

Each lesson serves as a kind of chess problem, in which 'check-mate' should be reached within a few, correct moves. This suggests a study process where thinking takes more time than the execution, which was the main concept in the development of the exercises. Any simplifications that have been introduced (neglected minor losses, pump curve definition, etc.) were meant to facilitate this process. In addition, the worksheets have been designed without complicated routines or macros; only a superficial knowledge of spreadsheets is required to be able to use them effectively.

This is the first edition and any suggestions for improvement or extension are obviously welcome.

N. Trifunović



Lesson 5-1 Gravity Supply – Residual Pressure

Contents:

Calculation of the friction losses in a gravity-fed system (application of the Darcy–Weisbach formula).

Goal:

Pressure analysis for various demands, locations of the reservoir and consumers' points, and change in pipe parameters.

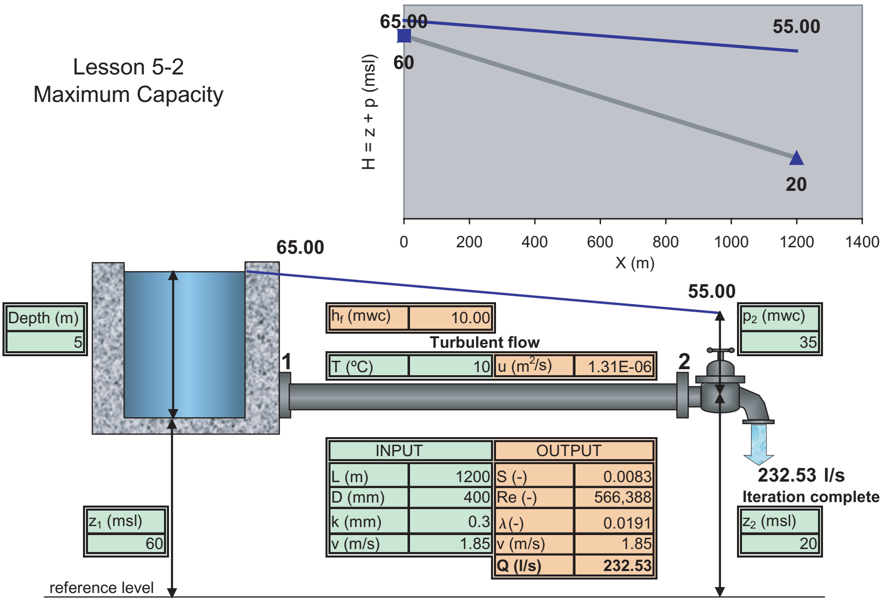
Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate
T (ºC)	Water temperature (degrees Celsius)
Depth (m)	Water depth in the reservoir
z_1 (msl)	Elevation of the reservoir bottom (metres above sea level)
z_2 (msl)	Elevation of the discharge point

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss (metres of water column)
S (-)	Hydraulic gradient
u (m^2/s)	Kinematic viscosity
p_2 (mwc)	Pressure at the discharge point

Notes:

The calculation ultimately yields the pressure at the discharge point. The graph shows the actual pipe route with its hydraulic grade line.



Lesson 5-2 Gravity Supply – Maximum Capacity

Contents:
Hydraulic calculation of a gravity-fed system.

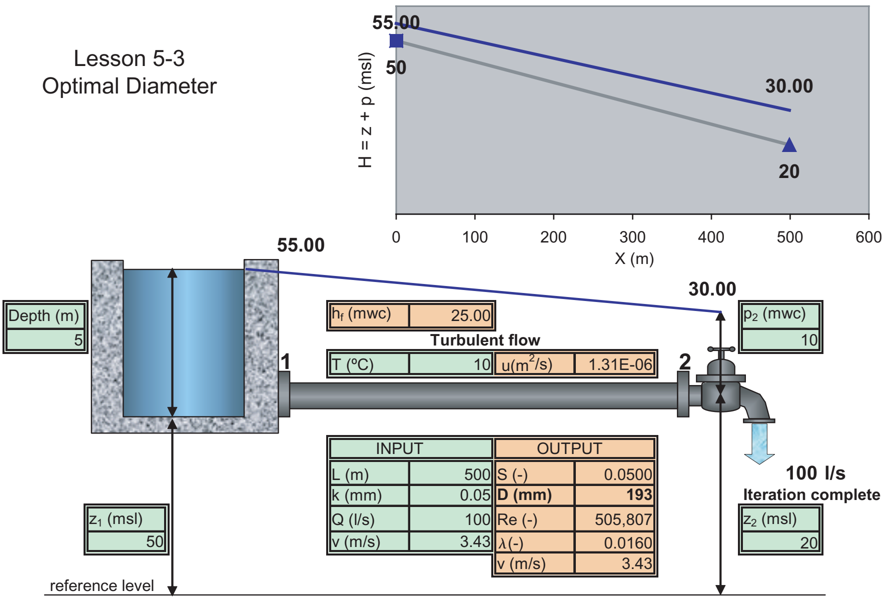
Goal:
Determination of the maximum discharge at the required minimum pressure and various positions of the reservoir.

Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate
T (°C)	Water temperature
Depth (m)	Water depth in the reservoir
z_1 (msl)	Elevation of the reservoir bottom
z_2 (msl)	Elevation of the discharge point
v (m/s)	Assumed flow velocity
p_2 (mwc)	Pressure required at the discharge point

S (-)	Hydraulic gradient
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
v (m/s)	Calculated flow velocity
Q (l/s)	Flow rate (discharge)
u (m^2/s)	Kinematic viscosity
h_f (mwc)	Friction loss

Notes:
The iterative procedure starts by assuming the flow velocity (commonly at 1 m/s) required for determination of the Reynolds number i.e. the friction factor.
The velocity calculated afterwards by the Darcy–Weisbach formula serves as an input for the next iteration.
The iterative process is achieved by typing the value of the calculated velocity into the cell of the assumed velocity.
The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.



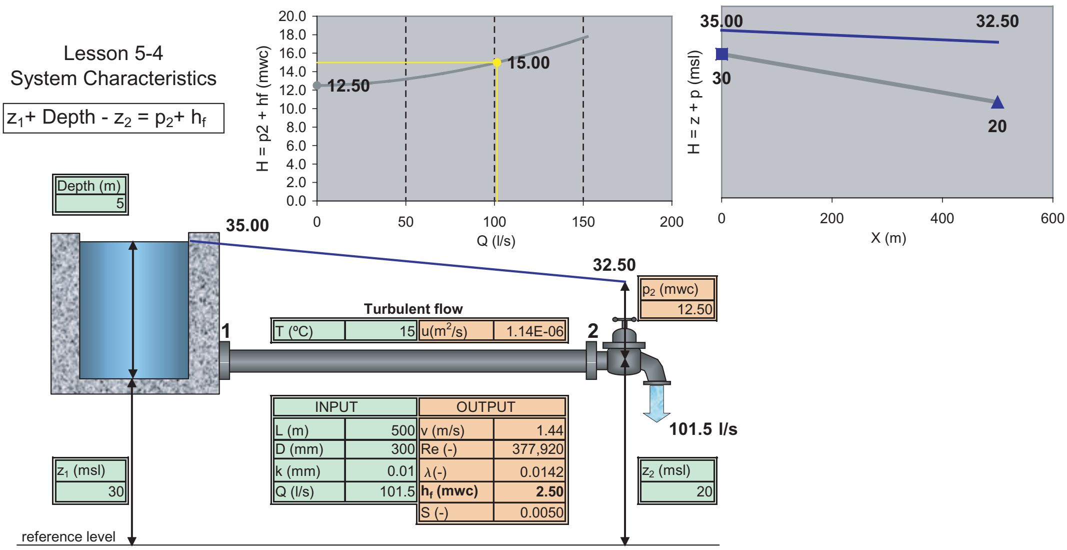
Lesson 5-3 Gravity Supply – Optimal Diameter

Contents:
Hydraulic calculation of a gravity-fed system.

Goal:
Determination of the optimal pipe diameter at the required minimum pressure and various positions of the reservoir.

Abbreviations:
The same as in Lesson 5-2.

Notes:
The same iterative procedure is used as in Lesson 5-2, except that the pipe diameter is determined from the assumed/calculated velocity (and specified flow rate). The message **Iteration complete** appears once the difference between the velocities in two iterations drops below 0.01 m/s.



Lesson 5-4 Gravity Supply – System Characteristics

Contents:

Friction loss calculation of a gravity-fed system.

Goal:

Determination of the system characteristics diagram.

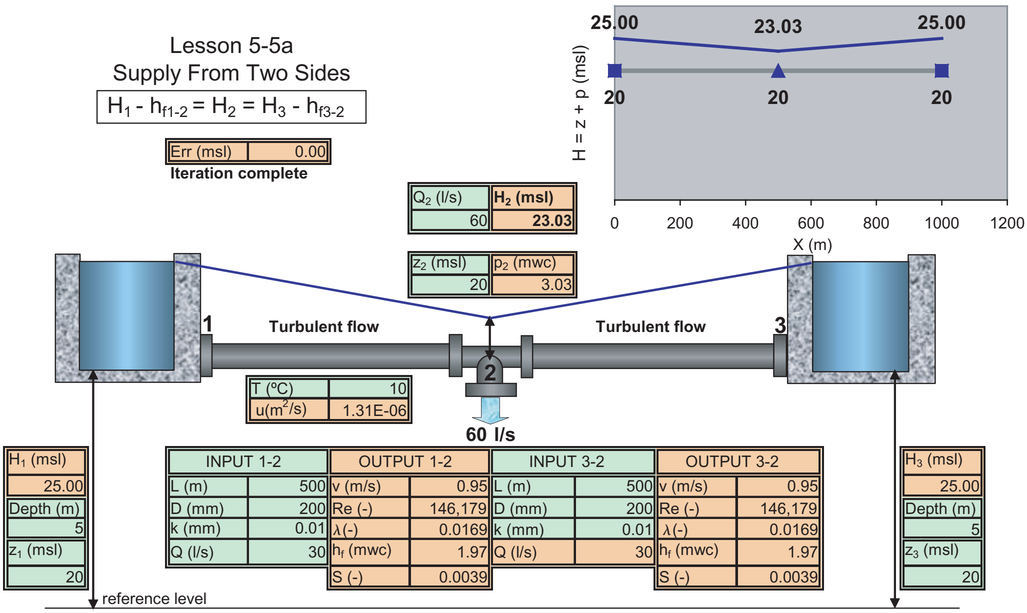
Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q (l/s)	Flow rate
T (ºC)	Water temperature
Depth (m)	Water depth in the reservoir
z_1 (msl)	Elevation of the reservoir bottom
z_2 (msl)	Elevation of the discharge point

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
S (-)	Hydraulic gradient
p_2 (mwc)	Pressure at the discharge point
u (m^2/s)	Kinematic viscosity

Notes:

The friction loss is calculated for the flow range 0–1.5Q (specified) and the results are plotted on the graph.
The point on the graph shows the upstream head required to maintain the downstream pressure for flow Q.
That head equals the elevation difference between the water surface in the reservoir and the discharge point.
The static head equals the downstream pressure, which fluctuates when the system parameters are modified.



Lesson 5-5a Gravity Supply – Supply from Two Sides

Contents:
Friction loss calculation of a gravity system fed from two sides.

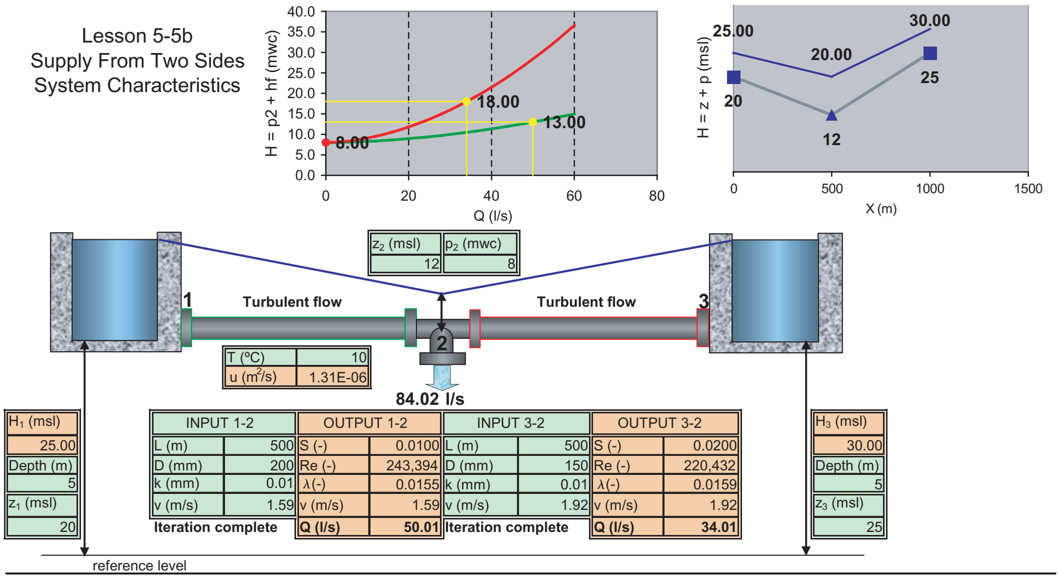
Goal:
To calculate the contribution from each source, based on various locations of the supply and discharge points, and changes in pipe parameters.

Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q_2 (l/s)	Discharge
Q (l/s)	Flow rate in pipe 1-2
T (ºC)	Water temperature
$z_{1,3}$ (msl)	Elevation of the reservoir bottoms
z_2 (msl)	Elevation of the discharge point
Depth (m)	Water depth in the reservoirs

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
S (-)	Hydraulic gradient
Q (l/s)	Flow rate in pipe 3-2
u (m^2/s)	Kinematic viscosity
$H_{1\text{-}3}$ (msl)	Piezometric heads
Err (msl)	The difference in H_2 calculated from both sides

Notes:
The process of trial and error consists of altering the flow in pipe 1-2, which has implications for the head-losses in both pipes.
The process ends when the difference in H_2 calculated from the left and right side drops below 0.01 msl and the message **Iteration complete** appears.
Negative values for pipe flows indicate a change of direction i.e. the water that is flowing to the reservoir.



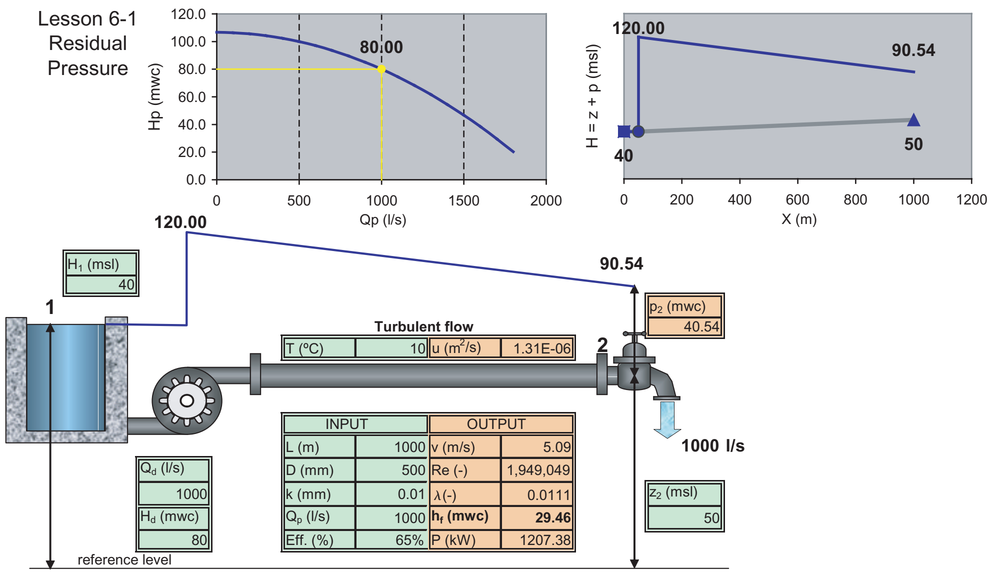
Lesson 5-5b Gravity Supply – Supply from Two Sides, System Characteristics

Contents:
Friction loss calculation of a gravity system fed from two sides.

Goal:
To calculate the contribution from each source, based on various locations of the supply and discharge points, and changes in pipe parameters.

Abbreviations:
The same as in Lesson 5-5a.

Notes:
The contribution from each source to the discharge is analysed by constructing pipe characteristics for both pipes.
The same procedure as in Lesson 5-4 is applied here.



Lesson 6-1 Pumped Supply – Residual Pressure

Contents:
Calculation of the friction losses in a pumped system.

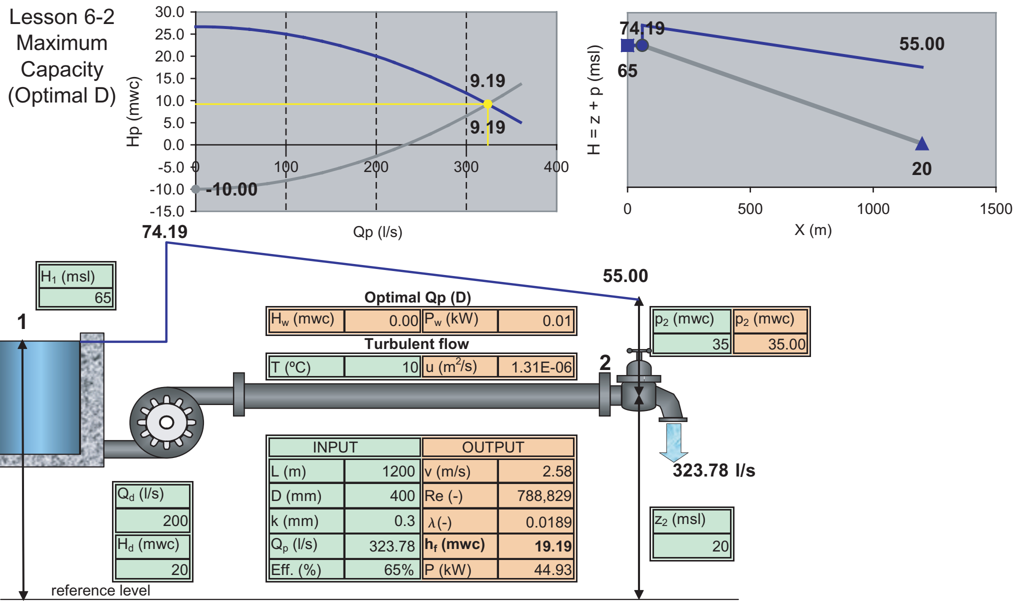
Goal:
Calculation of pumping heads and friction losses in a pumped system.

Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q_p (l/s)	Pumped discharge
Eff. (%)	Pump efficiency at Qp
Q_d (l/s)	Pump duty flow
H_d (mwc)	Pump duty head
H_1 (msl)	Piezometric head at the suction side of the pump
z_2 (msl)	Elevation of the discharge point
T (ºC)	Water temperature

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ(-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
P (k W)	Pump power consumption at Qp
p_2 (mwc)	Pressure at the discharge point
u (m^2/s)	Kinematic viscosity

Notes:
The pump curve is approximated by the formula $H_p = c - aQ_p^b$, where $c = 4H_d/3$, $b = 2$ and $a = H_d/3/Q_d^2$.
The pump graph is plotted for the Q_p range between 0 and $1.8Q_d$. The duty head and duty flow indicate the working point of the maximum pump efficiency.
The pump raises head H_1 to head $H_1 + H_p$ from where the pipe friction loss is going to be deducted.
The calculation ultimately yields the pressure at the discharge point. The graph shows the actual pipe route with its hydraulic grade line.



Lesson 6-2 Pumped Supply – Maximum Capacity/Optimal Diameter

Contents:

Calculation of pumping heads and friction losses in a pumped system.

Goal:

Relation between the pump and system characteristics.

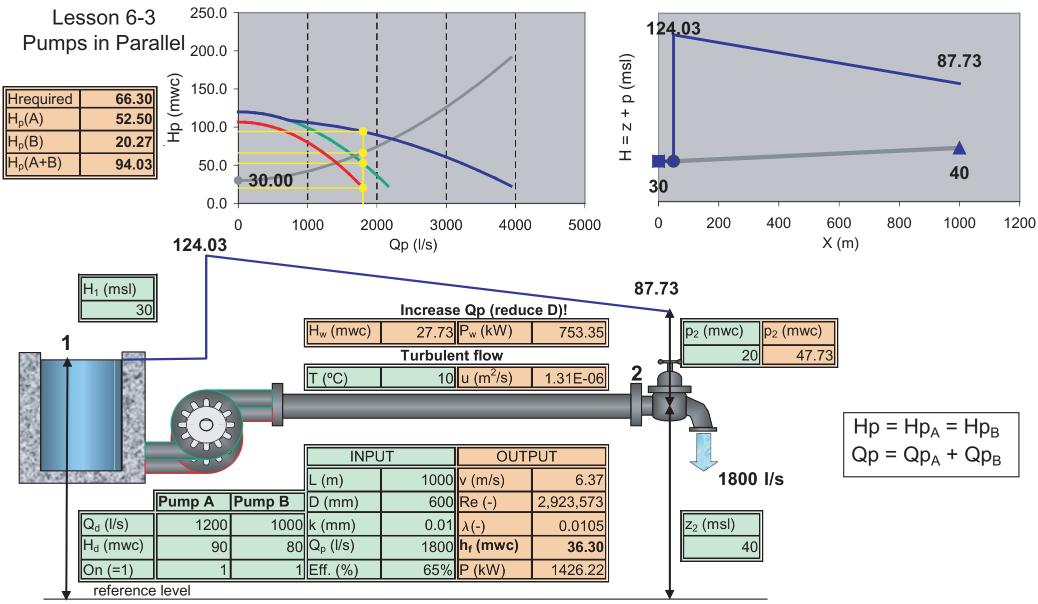
Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q_p (l/s)	Pumped discharge
Eff. (%)	Pump efficiency at Q_p
Q_d (l/s)	Pump duty flow
H_d (mwc)	Pump duty head
H_1 (msl)	Piezometric head at the suction side of the pump
z_2 (msl)	Elevation of the discharge point
p_2 (mwc)	Minimum pressure required at the discharge point
T (ºC)	Water temperature

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
P (kW)	Pump power consumption at Q_p
p_2 (mwc)	Actual pressure at the discharge point
H_w (mwc)	Excessive pumping head
P_w (kW)	Excessive pumping power
u (m^2/s)	Kinematic viscosity

Notes:

The pump curve is plotted in the same way as in Lesson 6-1. The system characteristics curve is plotted for the same range of flows.
The diagram shows the difference between the pumping head and the head required to deliver the minimum pressure at the discharge point.
The optimal working point is in the intersection between the pump- and system characteristics curves ($H_w = 0$).



Lesson 6-3 Pumped Supply – Pumps in Parallel

Contents:

Calculation of pumping heads and friction losses for parallel arrangement of the pumps.

Goal:

Relation between the pump- and system characteristics for various pump sizes and operational modes.

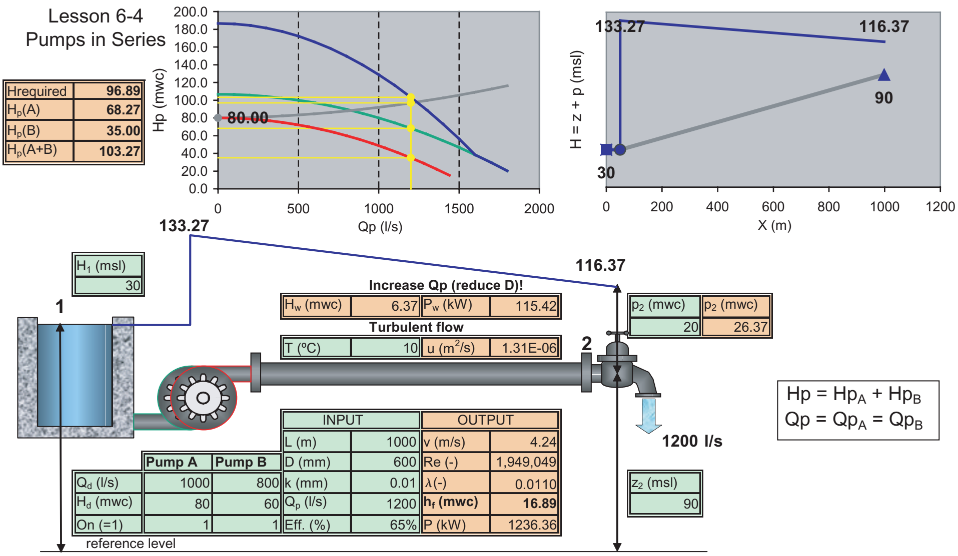
Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q_p (l/s)	Pumped discharge
Eff. (%)	Efficiency at Q_p
Q_d (l/s)	Pump duty flow (A,B)
H_d (mwc)	Pump duty head (A,B)
On (=1)	The pump is 'off' unless the cell input =1
H_1 (msl)	Piezometric head at the suction side of the pump
z_2 (msl)	Elevation of the discharge point
p_2 (mwc)	Minimum pressure required at the discharge point
T (ºC)	Water temperature

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
P (k W)	Pump power consumption at Q_p
p_2 (mwc)	Actual pressure at the discharge point
H_w (mwc)	Excessive pumping head
P_w (kW)	Excessive pumping power
u(m^2/s)	Kinematic viscosity
Hrequired	Minimum required head at Q_p
H_p(A)	Pumping head A at Q_p
H_p(B)	Pumping head B at Q_p
H_p(A+B)	Head of pumps A & B both in operation at Q_p

Notes:

The pump and system characteristics curves are plotted in the same way as in Lesson 6-2.
The diagram shows the difference between the pumping head and the head required to deliver the minimum pressure at the discharge point in all three cases: single operation of pumps A & B and their joint operation.
The hydraulic grade line on the graph is plotted for actual operation of the pumps.



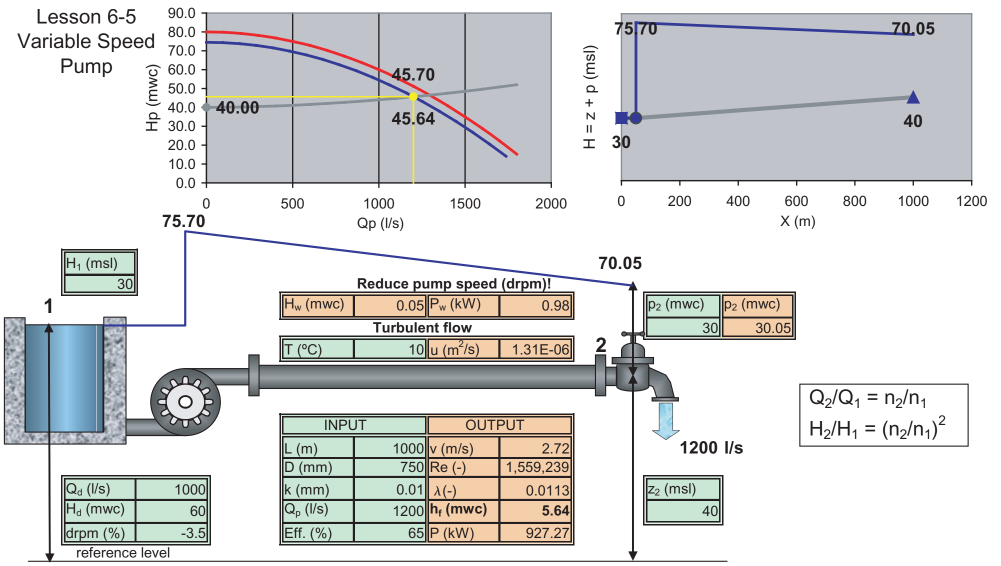
Lesson 6-4 Pumped Supply – Pumps in Series

Contents:
Calculation of pumping heads and friction losses for serial arrangement of pumps.

Goal:
Relation between the pump- and system characteristics curves for various pump sizes and operational modes.

Abbreviations:
The same as in Lesson 6-3.

Notes:
The same as in Lesson 6-3.



Lesson 6-5 Pumped Supply – Variable Speed Pump

Contents:

Calculation of pumping heads and friction losses in a system with variable speed pump.

Goal:

Analysis of effects caused by modification of the pump speed.

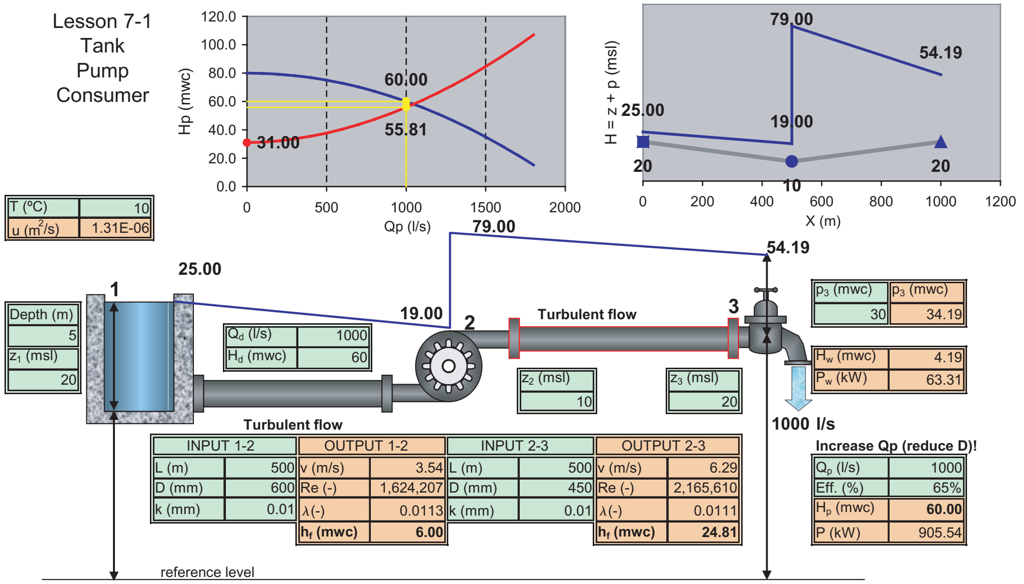
Abbreviations:

L (m)	Pipe length
D (mm)	Pipe diameter
k (mm)	Internal roughness
Q_p (l/s)	Pumped discharge
Eff. (%)	Pump efficiency at Q_p
Q_d (l/s)	Pump duty flow
H_d (mwc)	Pump duty head
drpm (%)	Pump speed increase/decrease
H_1 (msl)	Piezometric head at the suction side of the pump
z_2 (msl)	Elevation of the discharge point
p_2 (mwc)	Minimum pressure required at the discharge point
T (ºC)	Water temperature

v (m/s)	Flow velocity
Re (-)	Reynolds number
λ (-)	Darcy–Weisbach friction factor
h_f (mwc)	Friction loss
P (k W)	Pump power consumption at Q_p
p_2 (mwc)	Actual pressure at the discharge point
H_w (mwc)	Excessive pumping head
P_w (kW)	Excessive pumping power
u (m^2/s)	Kinematic viscosity

Notes:

The diagram shows the difference in pumping heads between the pump curve at the original and modified speeds.



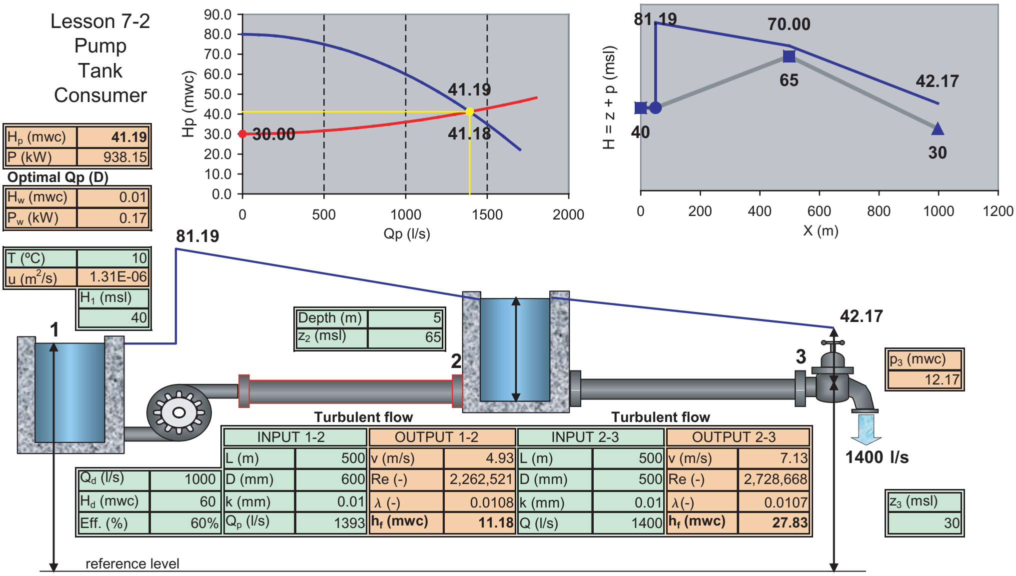
Lesson 7-1 Combined Supply – Tank, Pump, Consumer

Contents:
Calculation of pumping heads and friction losses in a system combining gravity and pumped supply.

Goal:
Determination of residual pressure/maximum capacity/optimal diameters for various scenarios.

Abbreviations:
The same as in Lessons 5 and 6.

Remarks:
The system characteristics curve in the diagram is plotted for the pipe on the pressure side of the pump.



Lesson 7-2 Combined Supply – Pump, Tank, Consumer

Contents:

Calculation of pumping heads and friction losses in a system combining the gravity and pumped supply.

Goal:

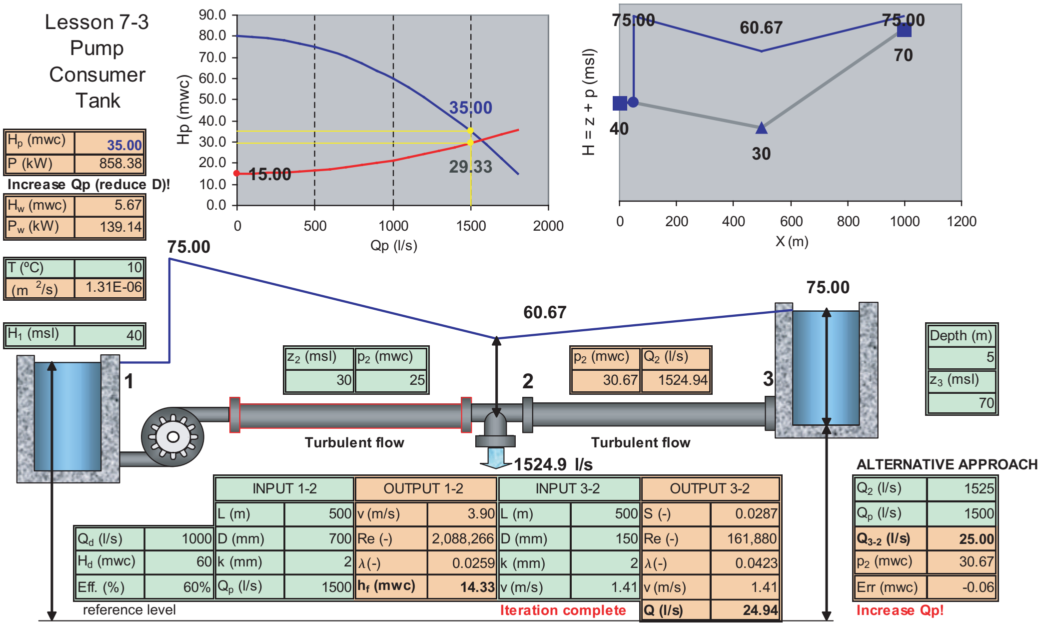
Determination of residual pressure/maximum capacity/optimal diameters for various scenarios.

Abbreviations:

The same as in Lessons 5 and 6.

Remarks:

The system characteristics in the diagram is plotted for the pipe on the upstream side of the reservoir.
The static head of that pipe is equal to the difference between the water levels in the two reservoirs.
The system is hydraulically disconnected and the maximum discharge capacity is dependant on the gravity part only.



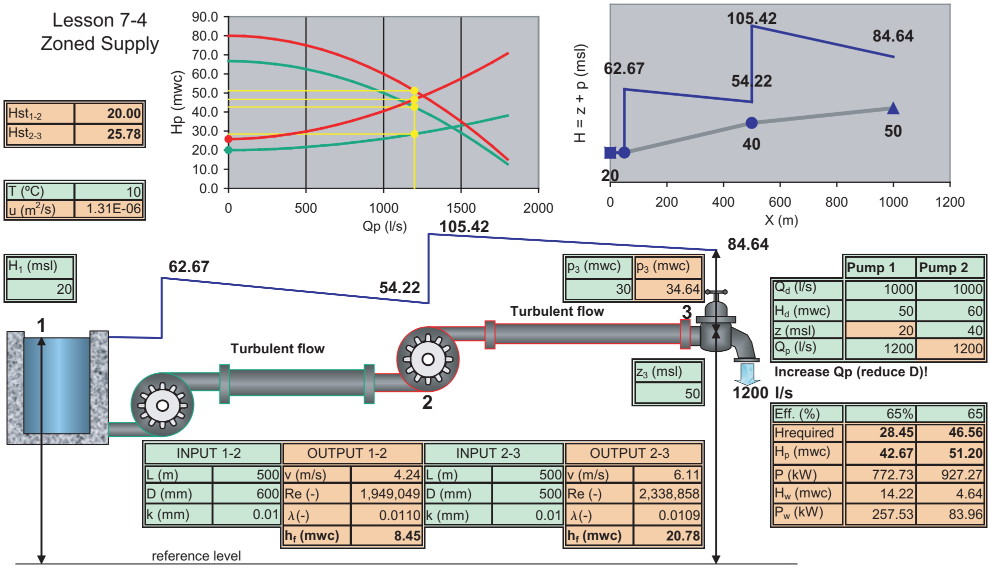
Lesson 7-3 Combined Supply – Pump, Consumer, Tank

Contents:
Calculation of pumping heads and friction losses in a system combining gravity and pumped supply.

Goal:
Determination of residual pressure/maximum capacity/optimal diameters for various scenarios.

Abbreviations:
The same as in Lessons 5 and 6.

Notes:
For a specified pumping flow, the contribution from the reservoir will be calculated through an iteration of velocities in pipe 3-2.
A negative value of the hydraulic gradient (S) in pipe 3-2 indicates a reversed flow direction i.e. filling of the reservoir (night regime).
By the alternative approach (see the table 'ALTERNATIVE APPROACH') the contribution from both supplying points will be determined for specified demand.
This is reached through a process of trial and error, with 'Err' indicating the difference in head H_2 calculated from both sides.
The pump is switched off by leaving the cells for H_d or/and Q_d empty.



Lesson 7-4 Combined Supply – Zoned Supply

Contents:
Calculation of pumping heads and friction losses in a system combining the gravity and pumped supply.

Goal:
Determination of residual pressure/maximum capacity/optimal diameters for booster pumping.

Abbreviations:
The same as in Lessons 5 and 6. In addition, '$Hst_{1\text{-}2}$' and '$Hst_{3\text{-}2}$' indicate the static heads of pipes 1-2 and 3-2 respectively.

Remarks:
The diagram shows the pump characteristics of each pump with its corresponding pipe characteristics.
The optimal working point for specified discharge and minimum downstream pressure is in the intersection of the red coloured curves.
The green coloured curves are monitored for avoiding under-pressure in the part of the system between the two pumps.
The pumps are switched off by leaving the cells for H_d or/and Q_d empty.

Lesson 8-1
Flow Conversion

Flow Rate	Units
100	l/s

converted into:

Units	Flow Rate
m3/h	360

VOLUME	m^3	litre	ft^3	gal	galUS
	1	2	3	4	5
m^3	1	1000	35.315	219.97	264.19
litre	0.0010	1	0.0353	0.2200	0.2642
ft^3	0.0283	28.317	1	6.2288	7.4811
gal	0.0045	4.5461	0.1605	1	1.2011
galUS	0.0038	3.7851	0.1337	0.8326	1

TIME	sec	min	hour	day
	1	2	3	4
sec	1	0.0167	2.8E-04	1.2E-05
min	60	1	0.0167	6.9E-04
hour	3600	60	1	0.0417
day	86400	1440	24	1

FLOW	m^3/s	l/s	m^3/h	Ml/d	ft^3/s	gpm	mgd	gpmUS	mgdUS
	1	2	3	4	5	6	7	8	9
m^3/s	1	1000	3600	86.4	35.315	13198	19.005	15852	22.826
l/s	0.001	1	3.6	0.0864	0.0353	13.198	0.019	15.852	0.0228
m^3/h	2.8E-04	0.2778	1	0.0240	0.0098	3.6661	0.005	4.4032	0.0063
Ml/d	0.0116	11.574	41.667	1	0.4087	152.76	0.220	183.47	0.2642
ft^3/s	0.0283	28.317	101.94	2.4466	1	373.73	0.538	448.87	0.6464
gpm	7.6E-05	0.0758	0.2728	0.0065	0.0027	1	0.001	1.2011	0.0017
mgd	0.0526	52.617	189.42	4.5461	1.8581	694.44	1	834.06	1.2011
gpmUS	6.3E-05	0.0631	0.2271	0.0055	0.0022	0.8326	0.0012	1	0.0014
mgdUS	0.0438	43.809	157.71	3.7851	1.5471	578.20	0.8326	694.44	1

Hour	Demand	
	l/s	m^3/h
1	989.6	3562.56
2	945.9	3405.24
3	902.2	3247.92
4	727.6	2619.36
5	844	3038.4
6	1164.2	4191.12
7	1571.7	5658.12
8	1600.8	5762.88
9	1775.4	6391.44
10	1964.6	7072.56
11	2066.4	7439.04
12	2110.1	7596.36
13	1600.8	5762.88
14	1309.7	4714.92
15	1091.4	3929.04
16	945.9	3405.24
17	1062.3	3824.28
18	1455.2	5238.72
19	1746.3	6286.68
20	2139.2	7701.12
21	2110.1	7596.36
22	2037.3	7334.28
23	1746.3	6286.68
24	1018.7	3667.32
Total	34,925.7	125,733
Average	1455.24	5238.86

Lesson 8-1 Flow Conversion

Contents:
Comparison of the flow units.

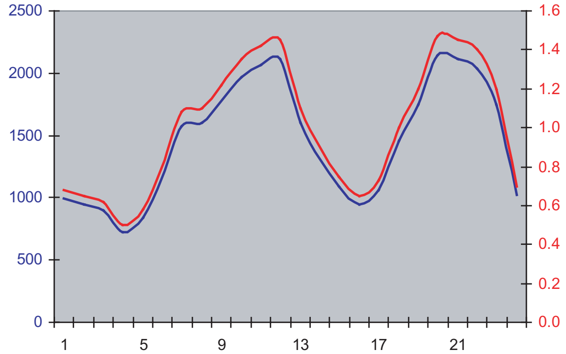
Goal:
Conversion of the common flow units.

Abbreviations:

m^3/s	cubic metre per second
l/s	litre per second
m^3/h	cubic metre per hour
Ml/d	mega-litre per day
ft^3/s	cubic feet per second
gpm	gallon per minute (Imperial)
mgd	mega-gallon per day (Imperial)
gpmUS	gallon per minute (US)
mgdUS	mega-gallon per day (US)

Notes:
Both individual values and the series of 24 hourly flows are converted from/to the units specified in the table (by typing).

Lesson 8-2
Diurnal Demand



	Hour	Demand (m^3/h)	Peak Factor
Maximum	20	2139.20	1.47
Minimum	4	727.60	0.50

Hour	Demand (m^3/h)	Peak Factor
1	989.60	0.680
2	945.90	0.650
3	902.20	0.620
4	727.60	0.500
5	844.00	0.580
6	1164.20	0.800
7	1571.70	1.080
8	1600.80	1.100
9	1775.40	1.220
10	1964.60	1.350
11	2066.40	1.420
12	2110.10	1.450
13	1600.80	1.100
14	1309.70	0.900
15	1091.40	0.750
16	945.90	0.650
17	1062.30	0.730
18	1455.20	1.000
19	1746.30	1.200
20	2139.20	1.470
21	2110.10	1.450
22	2037.30	1.400
23	1746.30	1.200
24	1018.70	0.700
Total	34,925.70	24.00
Average	1455.24	1.000

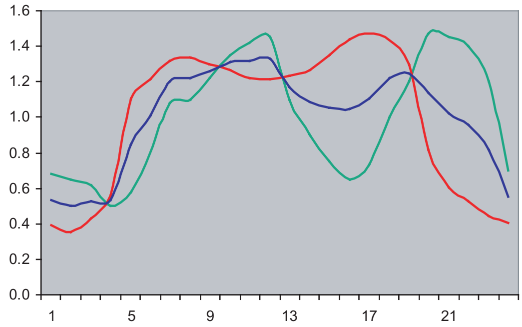
Lesson 8-2 Diurnal Demand

Contents:
Diurnal demand diagram.

Goal:
Determination of hourly peak factors.

Notes:
The peak factors are calculated as ratios between the actual and average hourly demand.

Lesson 8-3
Demand Categories



Peak Factors 1		Peak Factors 2		Cum. Peak Factors	
Maximum	1.470	Maximum	1.469	Maximum	1.332
at hour	20	at hour	17	at hour	12
Minimum	0.500	Minimum	0.351	Minimum	0.499
at hour	4	at hour	2	at hour	2

Hour	Cat.1 (m^3/h)	Peak Fact.1	Cat.2 (m^3/h)	Peak Fact.2	Demand (m^3/h)	Cum. Peak F.
1	989.60	0.680	579.00	0.389	1568.60	0.533
2	945.90	0.650	523.00	0.351	1468.90	0.499
3	902.20	0.620	644.00	0.432	1546.20	0.525
4	727.60	0.500	835.00	0.561	1562.60	0.531
5	844.00	0.580	1650.00	1.108	2494.00	0.847
6	1164.20	0.800	1812.00	1.217	2976.20	1.011
7	1571.70	1.080	1960.00	1.316	3531.70	1.200
8	1600.80	1.100	1992.00	1.338	3592.80	1.220
9	1775.40	1.220	1936.00	1.300	3711.40	1.261
10	1964.60	1.350	1887.00	1.267	3851.60	1.308
11	2066.40	1.420	1821.00	1.223	3887.40	1.320
12	2110.10	1.450	1811.00	1.216	3921.10	1.332
13	1600.80	1.100	1837.00	1.234	3437.80	1.168
14	1309.70	0.900	1884.00	1.265	3193.70	1.085
15	1091.40	0.750	2011.00	1.351	3102.40	1.054
16	945.90	0.650	2144.00	1.440	3089.90	1.049
17	1062.30	0.730	2187.00	1.469	3249.30	1.104
18	1455.20	1.000	2132.00	1.432	3587.20	1.218
19	1746.30	1.200	1932.00	1.297	3678.30	1.249
20	2139.20	1.470	1218.00	0.818	3357.20	1.140
21	2110.10	1.450	898.00	0.603	3008.10	1.022
22	2037.30	1.400	786.00	0.528	2823.30	0.959
23	1746.30	1.200	657.00	0.441	2403.30	0.816
24	1018.70	0.700	601.00	0.404	1619.70	0.550
Total	34,925.70	24.00	35,737.00	24.00	70,662.70	24.00
Average	1455.24	1.000	1489.04	1.000	2944.28	1.000

Lesson 8-3 Demand Categories

Contents:

Diurnal demand diagram.

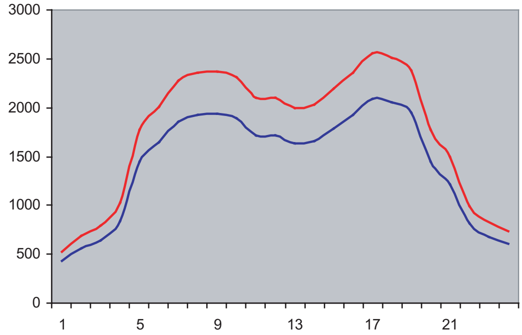
Goal:

Determination of hourly peak factors for various demand categories and the overall demand pattern.

Notes:

The peak factors of the cumulative demand diagram have no direct relation to the peak factors of the categories composing this diagram.

Lesson 8-4
Seasonal Variations



	Hour	Demand Day (m³/h) Current	Average	Peak Factors
Maximum	17	2087.00	2554.47	1.454
Minimum	1	433.00	529.99	0.302

D/M Hour	Sun Demand (m^3/h)	Oct Peak Factors	Average Demand (m^3/h)
1	433.00	0.302	529.99
2	562.00	0.391	687.88
3	644.00	0.449	788.25
4	835.00	0.582	1022.03
5	1450.00	1.010	1774.79
6	1644.00	1.145	2012.24
7	1856.00	1.293	2271.73
8	1922.00	1.339	2352.51
9	1936.00	1.349	2369.65
10	1887.00	1.314	2309.67
11	1721.00	1.199	2106.49
12	1712.00	1.193	2095.47
13	1634.00	1.138	2000.00
14	1656.00	1.154	2026.93
15	1789.00	1.246	2189.72
16	1925.00	1.341	2356.18
17	2087.00	1.454	2554.47
18	2055.00	1.431	2515.30
19	1944.00	1.354	2379.44
20	1453.00	1.012	1778.46
21	1218.00	0.848	1490.82
22	813.00	0.566	995.10
23	676.00	0.471	827.42
24	602.00	0.419	736.84
Average	1435.58	1.000	1757.14

Week	Peak Factors
Mon	1.140
Tue	1.010
Wed	0.950
Thu	0.980
Fri	1.040
Sat	1.020
Sun	0.860
Average	1.000

Year	Peak Factors
Jan	0.960
Feb	0.970
Mar	1.010
Apr	1.030
May	1.040
Jun	1.050
Jul	1.040
Aug	1.030
Sep	0.990
Oct	0.950
Nov	0.960
Dec	0.970
Average	1.000

Lesson 8-4 Seasonal Variations

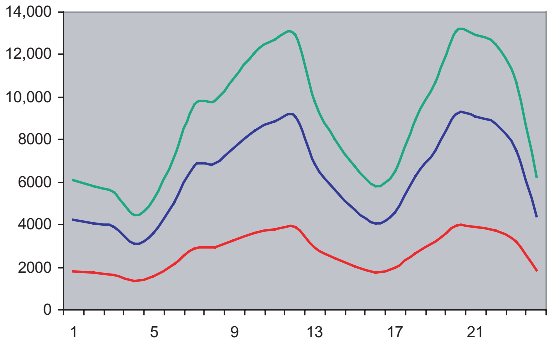
Contents:
Diurnal demand diagram.

Goal:
Comparison of the diurnal demand patterns for various periods of the week/year.

Abbreviations:
D/M Day/Month

Notes:
Recalculation of the hourly demand and corresponding peak factors takes place by typing the day and month from the tables for weekly and monthly peak factors.
The last peak factor value specified in the weekly and annual table is calculated so that the value can fill up to 7 and 12 respectively,
which generate the average equal to 1.0.

Lesson 8-5
Demand Calculation



Number of consum.	q (l/c/d)	$Q_{wc,\,avg}$ (m^3/h)	Leakage (%)	Equation 2.1/2.6	$Q_{d,\,avg}$ (m^3/h)
1,000,000	150	6250.00	30%	2.1	8928.57

Hour	Peak Factors	Water Quantities (m^3/h)		
		Q_{wc}	Q_{wl}	Q_d
1	0.680	4250.00	1821.43	6071.43
2	0.650	4062.50	1741.07	5803.57
3	0.620	3875.00	1660.71	5535.71
4	0.500	3125.00	1339.29	4464.29
5	0.580	3625.00	1553.57	5178.57
6	0.800	5000.00	2142.86	7142.86
7	1.080	6750.00	2892.86	9642.86
8	1.100	6875.00	2946.43	9821.43
9	1.220	7625.00	3267.86	10,892.86
10	1.350	8437.50	3616.07	12,053.57
11	1.420	8875.00	3803.57	12,678.57
12	1.450	9062.50	3883.93	12,946.43
13	1.100	6875.00	2946.43	9821.43
14	0.900	5625.00	2410.71	8035.71
15	0.750	4687.50	2008.93	6696.43
16	0.650	4062.50	1741.07	5803.57
17	0.730	4562.50	1955.36	6517.86
18	1.000	6250.00	2678.57	8928.57
19	1.200	7500.00	3214.29	10,714.29
20	1.470	9187.50	3937.50	13,125.00
21	1.450	9062.50	3883.93	12,946.43
22	1.400	8750.00	3750.00	12,500.00
23	1.200	7500.00	3214.29	10,714.29
24	0.700	4375.00	1875.00	6250.00
Average	1.000	6250.00	2678.57	8928.57

Lesson 8-5 Demand Calculation

Contents:
Diurnal demand diagram.

Goal:
Calculation of the leakage component for specified diurnal pattern and leakage percentage.

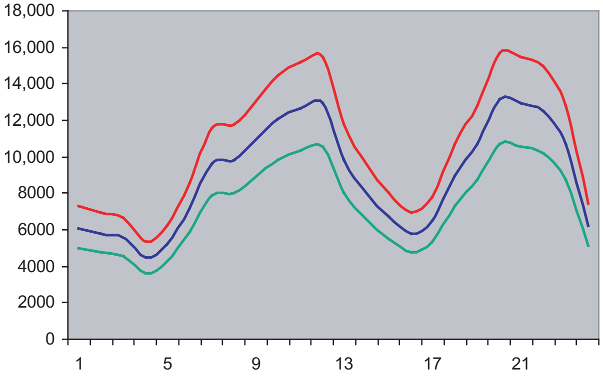
Abbreviations:

Number of consum.	Number of consumers in the area
q (l/c/d)	Specific consumption (unit per capita consumption)
$Q_{wc,\,avg}$ (m^3/h)	Average consumption
Leakage (%)	Leakage expressed as a percentage of water production
Equation 2.1/2.6	Equation used for the demand calculation, as discussed in Chapter 2
$Q_{d,\,avg}$ (m^3/h)	Average demand
Q_{wc} Q_{wl} Q_d	Hourly water consumption, water leakage and water demand, respectively

Notes:
The demand is calculated in two ways: a) with the leakage proportional to the peak factor values (Equation 2.1 in Chapter 2 of the book), and b) with constant leakage, independent from the peak factors (Equation 2.6). Both approaches yield the same average demand but the range of the hourly flows will be wider in the first approach. This adds a safety factor to the result although it reflects the reality to a lesser extent.
The last hourly peak factor value specified in the table is calculated so that the value can fill up to 24, which generates the average equal to 1.0.

Lesson 8-6
Design Demand

Number of consum.	q (l/c/d)	Leakage (%)	$Q_{d,\,avg}$ (m^3/h)
1,000,000	150	30%	8928.57



	Variation Peak Factors			Hour	Q_{peak}
	Weekday	Month	Overall		(m^3/h)
Maximum	1.140	1.050	1.197	20	15,710.63
Minimum	0.860	0.950	0.817	4	3647.32

Hour	Peak	Demand Day (m^3/h)		
	Factors	Avg.	Max.	Min.
1	0.680	6071.43	7267.50	4960.36
2	0.650	5803.57	6946.88	4741.52
3	0.620	5535.71	6626.25	4522.68
4	0.500	4464.29	5343.75	3647.32
5	0.580	5178.57	6198.75	4230.89
6	0.800	7142.86	8550.00	5835.71
7	1.080	9642.86	11,542.50	7878.21
8	1.100	9821.43	11,756.25	8024.11
9	1.220	10,892.86	13,038.75	8899.46
10	1.350	12,053.57	14,428.13	9847.77
11	1.420	12,678.57	15,176.25	10,358.39
12	1.450	12,946.43	15,496.88	10,577.23
13	1.100	9821.43	11,756.25	8024.11
14	0.900	8035.71	9618.75	6565.18
15	0.750	6696.43	8015.63	5470.98
16	0.650	5803.57	6946.88	4741.52
17	0.730	6517.86	7801.88	5325.09
18	1.000	8928.57	10,687.50	7294.64
19	1.200	10,714.29	12,825.00	8753.57
20	1.470	13,125.00	15,710.63	10,723.13
21	1.450	12,946.43	15,496.88	10,577.23
22	1.400	12,500.00	14,962.50	10,212.50
23	1.200	10,714.29	12,825.00	8753.57
24	0.700	6250.00	7481.25	5106.25
Average	1.000	8928.57	10,687.50	7294.64

Lesson 8-6 Design Demand

Contents:
Diurnal demand diagram.

Goal:
Establishing the range of demands/peak factors that can occur during the year.

Abbreviations:

Q_{peak} (m^3/h)	Absolute maximum and minimum peak demand

Notes:
Weekly and monthly factors compose the average consumption during the maximum/minimum consumption day. Maximum/minimum hourly peak factors in combination with the seasonal factors give the range of demands in the system between the maximum consumption hour of the maximum consumption day and the minimum consumption hour of the minimum consumption day.

The leakage component is calculated applying the first approach from the previous lesson (by using Equation 2.1).

The last hourly peak factor value specified in the table is calculated so that the value can fill up to 24, which generates the average equal to 1.0.

Lesson 8-7
Combined Demand

	CATEGORIES	q (m^3/d/ha)
A	Residential area, apartments	90
B	Individual houses	55
C	Shopping area	125
D	Offices	80
E	Schools, Universities	100
F	Hospitals	160
G	Hotels	150
H	Public green areas	15

	Q (m^3/h)	Inhab.	Q(l/c/d)
District 1	666.77	86,251	186
District 2	651.97	74,261	211
District 3	216.76	18,542	281
District 4	288.90	42,149	165
District 5	161.05	22,156	174
District 6	99.74	9958	240
District 7	58.03	8517	164
District 8	67.95	12,560	130
TOTAL	2211.16	274,394	193

		CONSUMPTION CATEGORIES							
		A	B	C	D	E	F	G	H
District 1	p (%)	37%	23%	10%	0%	4%	0%	0%	26%
Area 1 (ha)	c (%)	100%	100%	100%	0%	100%	0%	0%	40%
250	A_c(ha)	92.5	57.5	25	0	10	0	0	26
District 2	p (%)	20%	5%	28%	11%	12%	0%	5%	19%
Area 2 (ha)	c (%)	100%	100%	95%	100%	100%	0%	100%	80%
185	A_c(ha)	37	9.25	49.21	20.35	22.2	0	9.25	28.12
District 3	p (%)	10%	18%	3%	0%	0%	42%	0%	27%
Area 3 (ha)	c (%)	100%	100%	100%	0%	0%	100%	0%	35%
57	A_c(ha)	5.7	10.26	1.71	0	0	23.94	0	5.3865
District 4	p (%)	25%	28%	20%	2%	15%	0%	0%	10%
Area 4 (ha)	c (%)	100%	100%	95%	100%	100%	0%	0%	36%
88	A_c(ha)	22	24.64	16.72	1.76	13.2	0	0	3.168
District 5	p (%)	50%	0%	11%	0%	10%	0%	0%	29%
Area 5 (ha)	c (%)	100%	0%	100%	0%	100%	0%	0%	65%
54	A_c(ha)	27	0	5.94	0	5.4	0	0	10.179
District 6	p (%)	24%	11%	13%	15%	13%	8%	0%	16%
Area 6 (ha)	c (%)	100%	100%	100%	100%	100%	100%	0%	35%
29	A_c(ha)	6.96	3.19	3.77	4.35	3.77	2.32	0	1.624
District 7	p (%)	22%	28%	8%	19%	6%	0%	10%	7%
Area 7 (ha)	c (%)	100%	100%	100%	100%	100%	0%	100%	50%
17	A_c(ha)	3.74	4.76	1.36	3.23	1.02	0	1.7	0.595
District 8	p (%)	0%	0%	0%	0%	55%	20%	15%	10%
Area 8 (ha)	c (%)	0%	0%	0%	0%	85%	100%	100%	45%
16	A_c(ha)	0	0	0	0	7.48	3.2	2.4	0.72

Lesson 8-7 Combined Demand

Contents:

Calculation of the demand from the mixture of consumption categories in one district.

Goal:

To find out the total demand in the area based on the contribution from each consumption category and the network coverage in the districts.

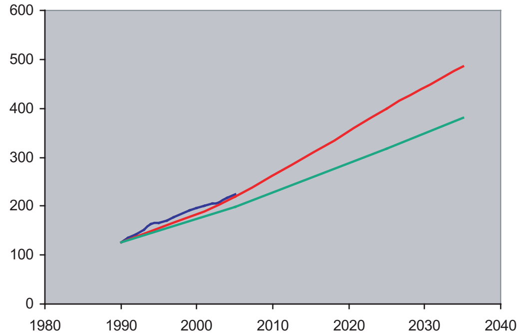
Abbreviations:

District 1	Name of the district
Area 1 (ha)	Total area of the district (in hectars)
p (%)	Percentage of the district area (surface) covered by a particular consumption category
c (%)	Percentage of the district area (surface) covered/supplied by the distribution network
A_c(ha)	Area of the district covered by the distribution network (in hectares)
Q (m^3/h)	Total demand in the district
Inhab.	Number of inhabitants in the district
Q(l/c/d)	Specific demand in the district

Notes:

An arbitrary list of categories and unit consumptions has been used.
A name can be used for each district; it copies itself into the table with the calculations.
Leakage can either be assumed as being included in the figure per category, or should be specified as a separate category.
The consumption figures indicated per category are assumed to be constant in all districts.
The contribution of the last category in each district is calculated so that the value can fill up to 100%, which represents the total district area.

Lesson 8-8
Demand Growth



Demand	
Year	Actual
1990	126.00
1991	135.00
1992	142.00
1993	151.00
1994	163.00
1995	166.00
1996	170.00
1997	177.00
1998	185.00
1999	192.00
2000	196.00
2001	201.00
2002	206.00
2003	208.00
2004	218.00
2005	225.00

Up to year	Growth (%)	Demand Forecast	
		Linear	Expon.
1990	-	126.00	126.00
2005	3.80%	197.82	220.46
2025	3.00%	316.51	398.18
2035	2.00%	379.81	485.37

Lesson 8-8 Demand Growth

Contents:
Demand forecast from the given historical data.

Goal:
Fitting of the actual growth pattern with the linear and exponential model.

Notes:
The first value and the corresponding year of the actual pattern are taken as the starting point in both models.
Time intervals in the tables can differ but the empty 'Year'-cells are allowed only at the end of the series.
The shorter time intervals make less difference between the results of the two models.

Lesson 8-9
Demand Frequency

Averege Day

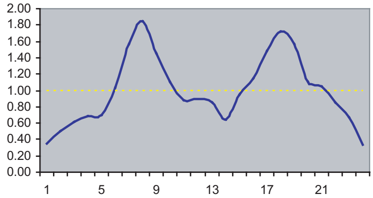
Hourly Peak	Hour	Peak Factors
Maximum	8	1.850
Minimum	24	0.340

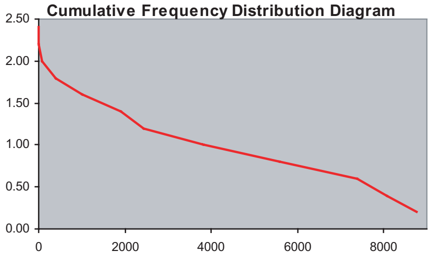
Peak Consumption Days

Hourly Peak	Sun Oct	Mon Jun
Maximum	1.511	2.214
Minimum	0.278	0.407

Cumulative Frequency

Peak Factor	Hours Per Year	Total Per Year
2.40	0	0%
2.20	4	0%
2.00	87	1%
1.80	396	5%
1.60	1006	11%
1.40	1907	22%
1.20	2448	28%
1.00	3822	44%
0.80	5589	64%
0.60	7381	84%
0.40	8069	92%
0.20	8760	100%





Hour	Peak Factors
1	0.350
2	0.500
3	0.620
4	0.680
5	0.700
6	1.030
7	1.560
8	1.850
9	1.460
10	1.100
11	0.880
12	0.900
13	0.850
14	0.650
15	0.950
16	1.150
17	1.480
18	1.720
19	1.560
20	1.100
21	1.050
22	0.850
23	0.670
24	0.340
Average	1.000

	Week	Peak Factors
	Mon	1.140
	Tue	1.010
	Wed	0.950
	Thu	0.980
	Fri	1.040
1[st]Jan	Sat	1.020
	Sun	0.860
	Average	1.000

Year	Peak Factors
Jan	0.960
Feb	0.970
Mar	1.010
Apr	1.030
May	1.040
Jun	1.050
Jul	1.040
Aug	1.030
Sep	0.990
Oct	0.950
Nov	0.960
Dec	0.970
Average	1.000

Lesson 8-9 Demand Frequency

Contents:

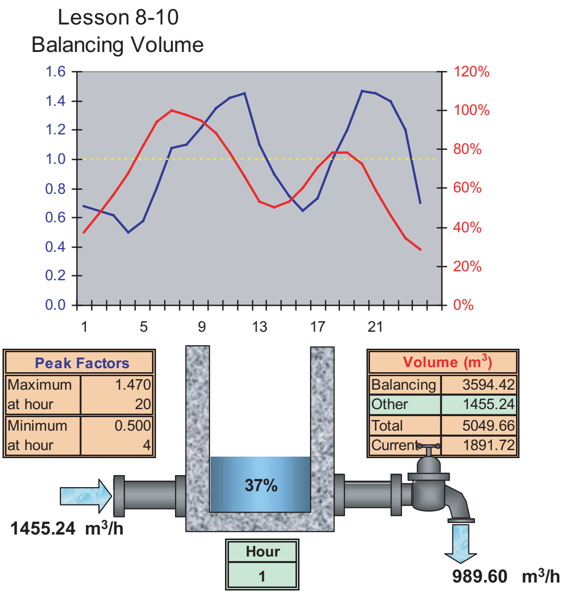
Calculation of the demand frequency distribution for given diurnal-, weekly and annual demand pattrens.

Goal:

Determination of the cumulative frequency diagram that should help to adopt the design peak factor.

Notes:

Maximum/minimum hourly peak factors in combination with the seasonal factors give the range of absolute values for the hourly peak factors in the system between the maximum consumption hour of the maximum consumption day and the minimum consumption hour of the minimum consumption day.
The last peak factor value specified in each input table is calculated so that the value can fill up to 24, 7 and 12 respectively, which generates the average equal to 1.0.
After the calculation of the absolute values, all peak factor values are compared and counted. The figures in the Cumulative Frequency table indicate the number of hours/percentage of the total time, when the absolute peak factor was above the corresponding value in the table.
The cell filled with **1[st]Jan** should be cut and pasted next to the week day which was assumed on January 1.



Hour	Tank In (m^3/h)	Tank Out (m^3/h)	Peak Factor	In-Out (m^3/h)	Cum. (m^3/h)	Volume
1	1455.24	989.60	0.680	465.64	465.64	37%
2	1455.24	945.90	0.650	509.34	974.98	47%
3	1455.24	902.20	0.620	553.04	1528.02	57%
4	1455.24	727.60	0.500	727.64	2255.66	68%
5	1455.24	844.00	0.580	611.24	2866.90	82%
6	1455.24	1164.20	0.800	291.04	3157.94	94%
7	1455.24	1571.70	1.080	-116.46	3041.48	100%
8	1455.24	1600.80	1.100	-145.56	2895.92	98%
9	1455.24	1775.40	1.220	-320.16	2575.76	95%
10	1455.24	1964.60	1.350	-509.36	2066.40	88%
11	1455.24	2066.40	1.420	-611.16	1455.24	78%
12	1455.24	2110.10	1.450	-654.86	800.38	66%
13	1455.24	1600.80	1.100	-145.56	654.82	53%
14	1455.24	1309.70	0.900	145.54	800.36	50%
15	1455.24	1091.40	0.750	363.84	1164.20	53%
16	1455.24	945.90	0.650	509.34	1673.54	61%
17	1455.24	1062.30	0.730	392.94	2066.48	71%
18	1455.24	1455.20	1.000	0.04	2066.52	78%
19	1455.24	1746.30	1.200	-291.06	1775.46	78%
20	1455.24	2139.20	1.470	-683.96	1091.50	73%
21	1455.24	2110.10	1.450	-654.86	436.64	59%
22	1455.24	2037.30	1.400	-582.06	-145.42	46%
23	1455.24	1746.30	1.200	-291.06	-436.48	35%
24	1455.24	1018.70	0.700	436.54	0.06	29%
Average	1455.24	1455.24	1.000	0.00		

Lesson 8-10 Balancing Volume

Contents:

The diurnal demand diagram and volume variation of the balancing reservoir.

Goal:

The determination of the balancing volume in the reservoir and its variation during the day.

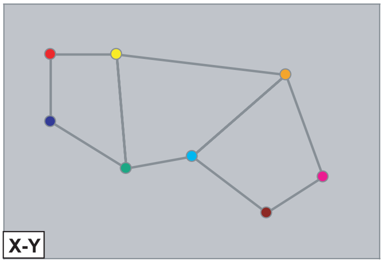
Notes:

The reservoir for which the volume is calculated is the only one existing in the system, and it is assumed that it balances the demand of the entire area. In the case of favourable topography, the total balancing volume can be shared between a few smaller reservoirs.

Lesson 8-11
Nodal Demands

$Q1_{tot}$ (l/s)	$Q2_{tot}$ (l/s)	$Q3_{tot}$ (l/s)	Q_{tot} (l/s)
120	180	65	365

NODES	X (m)	Y (m)	QL_1 (l/s)	QL_2 (l/s)	QL_3 (l/s)	Q_n (l/s)
n1	25	135	35.38	0.00	0.00	35.38
n2	25	200	30.57	0.00	0.00	30.57
n3	60	200	24.62	54.81	0.00	79.43
n4	65	88	29.43	40.03	0.00	69.46
n5	100	100	0.00	35.19	16.98	52.17
n6	150	180	0.00	49.97	22.04	72.01
n7	170	80	0.00	0.00	15.52	15.52
n8	140	45	0.00	0.00	10.46	10.46
n9			0.00	0.00	0.00	0.00
n10			0.00	0.00	0.00	0.00



PIPES	Node 1	Node 2	L_{xy} (m)	L (m)	Q (l/s)
p1	n1	n2	65	680	38.49
p2	n2	n3	35	400	22.64
p3	n3	n4	112	470	26.60
p4	n4	n1	62	570	32.26
					0.00
LOOP 1			Sum=	2120	120.00

PIPES	Node 1	Node 2	L_{xy} (m)	L (m)	Q (l/s)
p3	n4	n3	112	1120	60.18
p5	n3	n6	92	920	49.43
p6	n6	n5	94	940	50.51
p7	n5	n4	37	370	19.88
					0.00
LOOP 2			Sum=	3350	180.00

PIPES	Node 1	Node 2	L_{xy} (m)	L (m)	Q (l/s)
p6	n5	n6	94	940	21.14
p8	n6	n7	102	1020	22.94
p9	n7	n8	46	360	8.10
p10	n8	n5	68	570	12.82
					0.00
LOOP 3			Sum=	2890	65.00

Lesson 8-11 Nodal Demand

Contents:
Spatial demand distribution.

Goal:
Calculation of nodal demands based on simplified procedure for concentrating the demand into the junctions between the pipes.

Abbreviations:

NODES	Node data
X (m)	Horizontal co-ordinate
Y (m)	Vertical co-ordinate
QL_1 (l/s)	Contribution to the nodal demand from loop 1
QL_2 (l/s)	Contribution to the nodal demand from loop 2
QL_3 (l/s)	Contribution to the nodal demand from loop 3
Q_n (l/s)	Average (baseline) nodal demand

PIPES	Pipe data per loop
Node 1	Pipe node nr.1
Node 2	Pipe node nr.2
L_{xy} (m)	Length calculated from the X/Y co-ordinates
L (m)	Length adopted for hydraulic calculation
Q (l/s)	Flow supplied from the pipe to the loop

Notes:
The nodes are plotted based on the X/Y input (the origin of the graph is at the lower left corner). Any node name can be used.
The pipes are plotted based on the Node 1/Node 2 input. This input determines the connection between the nodes and hence the flow rates/directions.
Half of the pipe flow (Q) is allocated to each of the corresponding nodes.

APPENDIX 6

EPANET – Version 2

(based on the EPANET 2 Users Manual by L.A. Rossman[1])

A6.1 INSTALLATION

EPANET 2 is a computer programme that performs hydraulic and water quality simulations of drinking water distribution systems. For a basic set of input data related to the geometry of the network and the water demand levels in it, the programme is able to determine the flow of water in each pipe, the pressure at each pipe junction, the flows and heads at each pump, and the water depth in each storage tank. Additional information on water quality parameters serves to calculate the concentration of a substance throughout a distribution system. In addition to substance concentrations, water age and source tracing can also be simulated. The user is able to edit EPANET 2 files and, after running a simulation, display the results on a colour-coded map of the distribution system and generate additional tabular and graphical views of these results.

The calculations made by EPANET can help in solving all sorts of practical problems, such as:

- the design of new extensions and pumping stations,
- the analysis of pumps' energy and cost,
- the optimal operation which can guarantee delivery of sufficient water quality and pressure,
- the assessment of network reliability,
- the development of effective flushing programmes,
- the use of satellite treatment, such as re-chlorination at storage tanks,
- the diagnosis of water quality problems, etc.

EPANET 2 software (EN2setup.exe) and its manual (EN2manual.pdf) are in the public domain and can be downloaded from the website of the US Environmental Protection Agency. This site is easily accessible through any search engine (e.g. 'Google'), by using the keyword 'epanet'.

[1] Rossman, L.A., EPANET 2 Users Manual, EPA/600/R-00/57, Water Supply and Water Resources Division, U.S. Environmental Protection Agency, Cincinnati, OH, September 2000).

The file EN2setup.exe contains a self-extracting setup programme. The installation starts after double-clicking on this file. The default programme directory is *c:\Program Files\EPANET2*, which can be changed during the process that will be fully completed in just a few seconds. After the programme has been installed, the Start Menu will have a new item named EPANET 2.0. To start the programme, choose **Start** >> **Programs** >> **Epanet 2.0** >> **Epanet 2.0**.

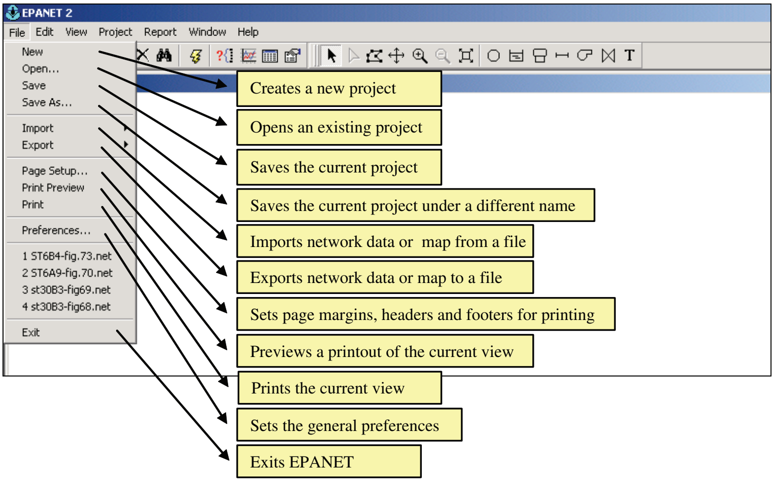
A6.2 USING THE PROGRAMME

After running the programme, a screen with the menu bar, toolbars, blank network map window and the browser window will appear.

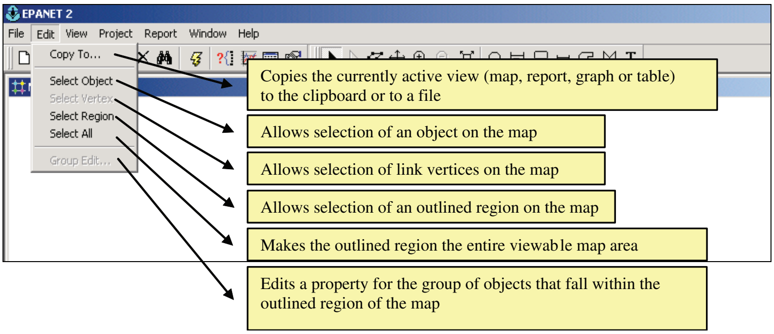
Menu bar

The *Menu bar* located across the top of the EPANET workspace contains a collection of menus used to control the program. These include:

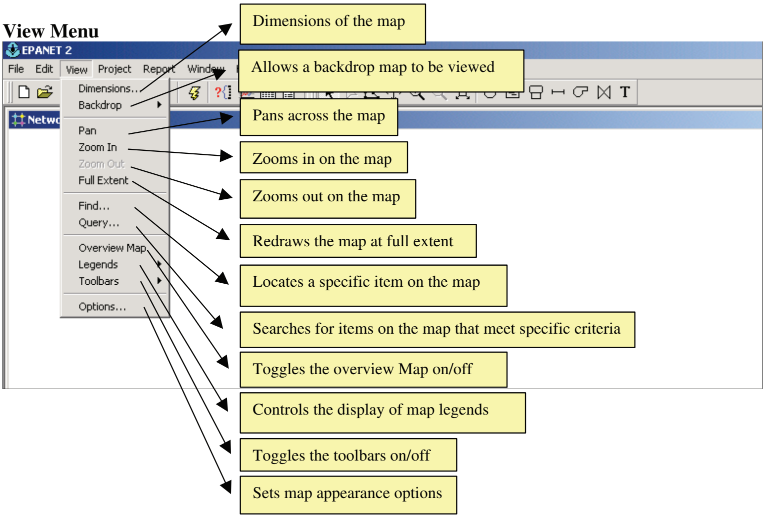
File Menu



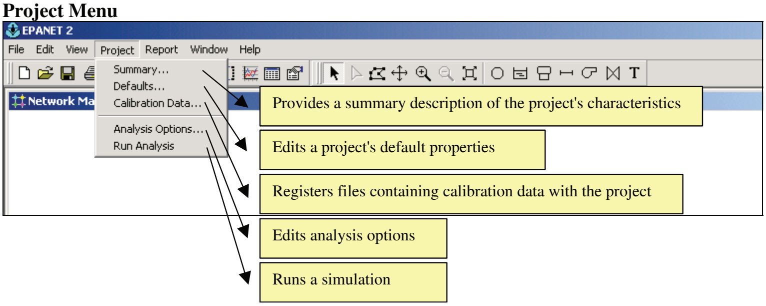
Edit Menu



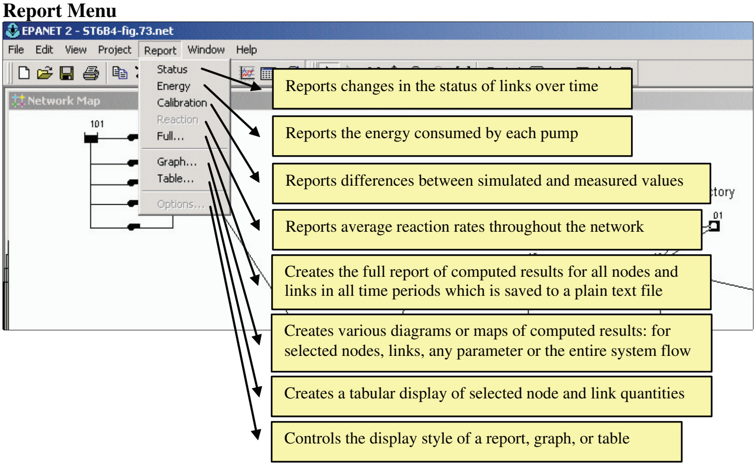
View Menu



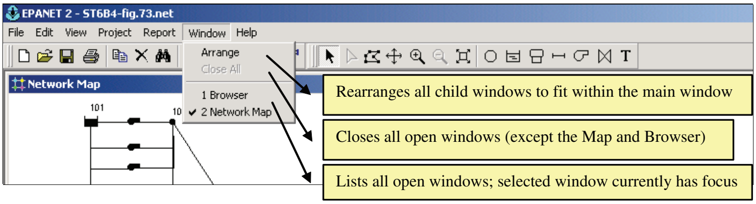
Project Menu



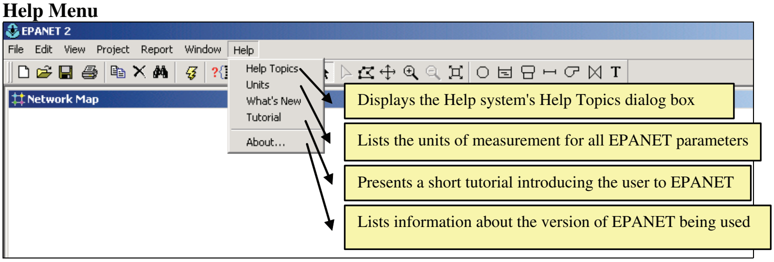
Report Menu



Window Menu



Help Menu



Toolbars

The *Toolbars* located below the Menu Bar provide shortcuts to commonly used operations. The toolbars can be made visible or invisible by selecting **View** >> **Toolbars**.

There are two such toolbars:

Standard toolbar

Standard toolbar: contains speed buttons for commonly used commands.

 Opens a new project (**File>>New**)

 Opens an existing project (**File>>Open**)

 Saves the current project (**File>>Save**)

 Prints the currently active window (**File>>Print**)

 Copies selection to the clipboard or to a file (**Edit>>Copy To**)

 Deletes the currently selected item

 Finds a specific item on the map (**View>>Find**)

 Runs a simulation (**Project>>Run Analysis**)

 Runs a visual query on the map (**View>>Query**)

 Creates a new graph view of results (**Report>>Graph**)

 Creates a new table view of results (**Report>>Table**)

 Modifies options for the currently active view (**View>>Options** or **Report>>Options**)

Map toolbar

Map toolbar: contains buttons for working with the Network Map.

 Selects an object on the map (**Edit>>Select>>Object**)

 Selects link vertex points (**Edit>>Select Vertex**)

 Selects a region on the map (**Edit>>Select Region**)

 Pans across the map (**View>>Pan**)

 Zooms in on the map (**View>>Zoom In**)

 Zooms out on the map (**View>>Zoom Out**)

 Draws map at full extent (**View>>Full Extent**)

 Adds a junction to the map

 Adds a reservoir to the map (fixed surface level, unknown dimensions)

 Adds a tank to the map (variable surface level, known dimensions)

 Adds a pipe to the map

 Adds a pump to the map

 Adds a valve to the map

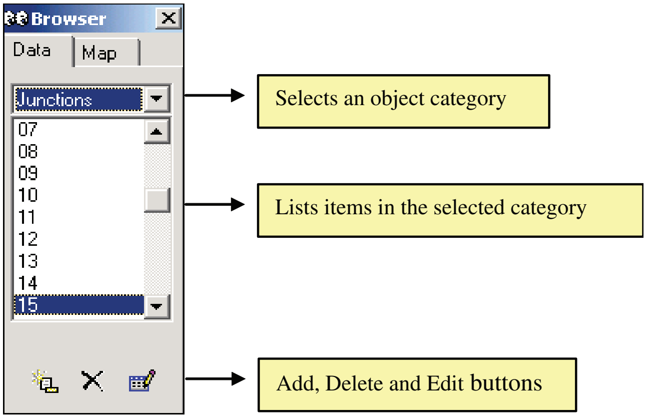
 Adds a label to the map

Network map

The *Network map* provides a planar schematic diagram of the objects comprising a water distribution network. The location of objects and the distances between them do not necessarily have to conform to their actual physical scale. New objects can be directly added to the map and existing objects can be clicked on for editing, deleting, and repositioning. A backdrop drawing (such as a street or topographic map) can be placed behind the network map for reference. The map can be zoomed to any scale and panned from one position to another. Nodes and links can be drawn at different sizes, flow direction arrows added and object symbols, ID labels and numerical property values displayed. The map can be printed, copied onto the Windows clipboard, or exported as a DXF file or Windows metafile (WMF).

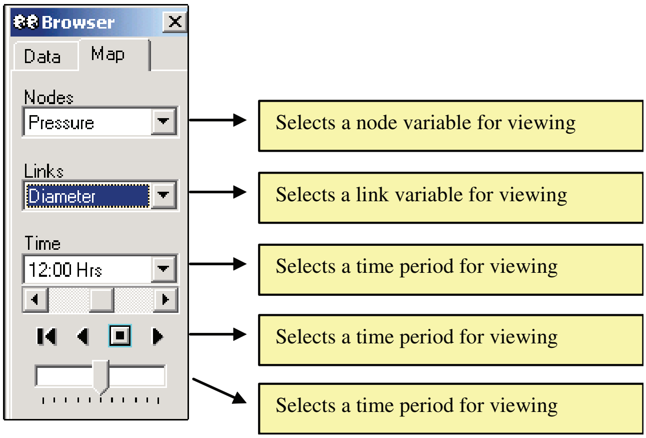
Data browser

The *Data browser* is accessed from the Data tab on the Browser window. It gives access to the various objects that exist in the network being analysed. The buttons at the bottom are used to add, delete and edit these objects.



Map browser

The *Map browser* is accessed from the Map tab of the Browser Window. It selects the parameters and time period that are viewed in colour-coded fashion on the Network Map. It also contains controls for animating the map through time.



The work begins by opening a network file, *.NET, by the menu option: **File** >> **Open** from the Menu Bar or clicking the Standard Toolbar  (If the toolbar is not visible **View** >> **Toolbars** >> **Standard** should selected be from the menu bar).

The selected file will be loaded into the computer memory. It is possible to choose to open a file type saved previously as an EPANET project (typically with a .NET extension) or exported as a text file (typically with a .INP extension). EPANET recognizes file types by their content, not their names.

Calculation starts by running the menu option: **Project** >> **Run Analysis** or clicking the Run button on the Standard Toolbar.

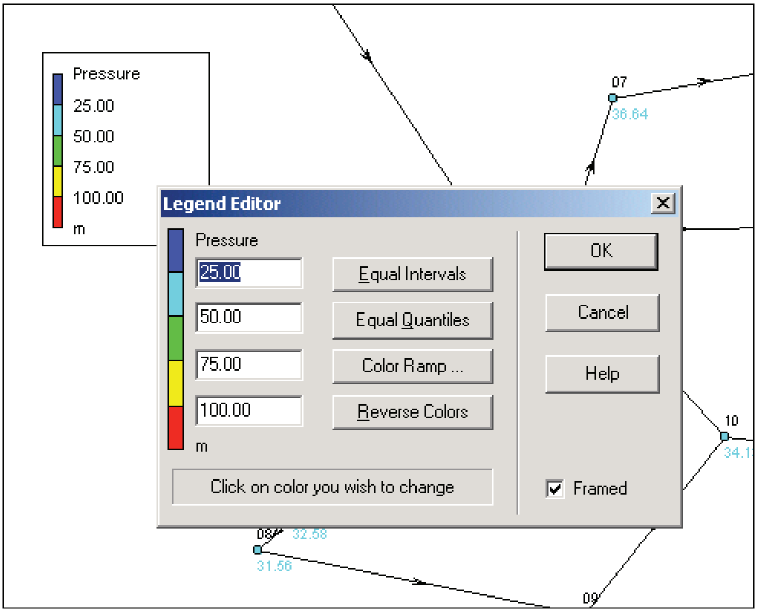
Status report

If the run was unsuccessful then a *Status report* window will appear indicating what the problem was. In some situations, the calculation may be completed regardless of the hydraulic boundary conditions. A warning message is displayed in that case. The input data have to be carefully analysed. More information about the calculation progress can be requested in repeated trials. The details about this, together with the description of other programme features, are presented in the full version of the programme manual.

If it ran successfully, the user can view the computed results in a variety of ways:

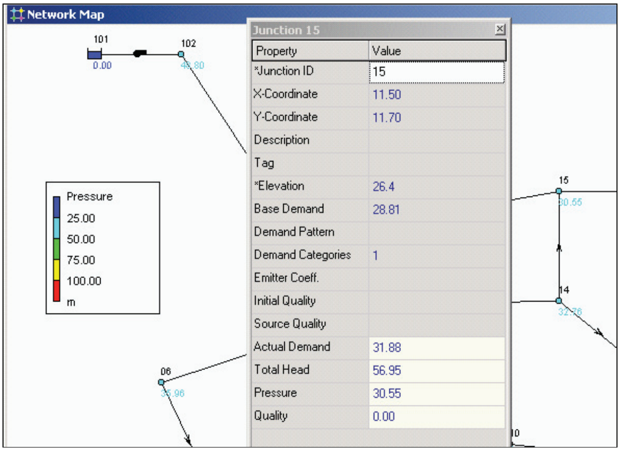
Legend editor

– Select Node Pressure from the Browser's Map page and observe how pressure values at the nodes become colour-coded. To view the legend for the colour coding, the user can select **View** >> **Legends** >>**Node** (or right click on an empty portion of the map and select **Node Legend** from the pop-up menu). To change the legend intervals and colours, right click on the legend makes the *Legend editor* appear.



Property editor

– The user can bring up the *Property editor* (double-click on any node or link) and note how the computed results are displayed at the end of the property list.



– A tabular listing of results is created by selecting **Report** >> **Table** (or by clicking the Table button on the Standard Toolbar). It is possible to create the report for all nodes or links for a specific time, or a report for a specific node or link for a time series. In the columns, it is possible to have a report for elevation, base demand, initial quality, demand, head, pressure and quality in the case of a node; and length, diameter, roughness, bulk coefficient, wall coefficient, flow, velocity, unit head loss, friction factor, reaction rate, quality and status in the case of a link.

Network Table - Nodes at 0:00 Hrs

Node ID	Demand LPS	Head m	Pressure m
Junc 01	0.00	56.74	35.74
Junc 02	14.52	59.20	41.40
Junc 03	34.31	58.26	38.76
Junc 04	38.69	57.62	37.82
Junc 05	37.94	54.98	32.58
Junc 06	20.84	55.96	35.96
Junc 07	29.85	57.84	36.64
Junc 08	11.92	54.46	31.56
Junc 09	37.21	52.24	27.54

A6.3 INPUT DATA

A6.3.1 *Data preparation*

Prior to running EPANET, the following initial steps should be taken for the network being studied:

1 All network components and their connections should be identified. Network components consist of pipes, pumps, valves, storage tanks and reservoirs. The term 'node' denotes a junction where network components connect to one another. Tanks and reservoirs are also

considered as nodes. The component (pipe, pump or valve) connecting any two nodes is termed a 'link'.

2 Unique ID numbers should be assigned to all nodes. ID numbers must be between 1 and 2,147,483,647, but need not to be in any specific order nor be consecutive.
3 An ID number should be assigned to each link (pipe, pump or valve). It is permissible to use the same ID number for both a node and a link.
4 The following information should be collected on the system parameters:
   a diameter, length, roughness and minor loss coefficient for each pipe,
   b the characteristic operating curve for each pump,
   c diameter, minor loss coefficient and pressure or flow setting for each control valve,
   d diameter and minimum and maximum water depths for each tank,
   e control rules that determine how pump, valve and pipe settings change with time, tank water levels or nodal pressures,
   f changes in water demands for each node over the time period being simulated,
   g initial water quality at all nodes and changes in water quality over time at source nodes.

With this information at hand, it is now possible to construct an input file to use with EPANET.

A6.3.2 *Selecting objects*

To select an object on the map:

Selection mode

– The map must be in *Selection mode* (the mouse cursor has the shape of an arrow pointing up to the left). To switch to this mode, the user should either click the Select Object button  on the Map Toolbar or choose **Edit** >>**Select Object** from the menu bar.
– The mouse has to be clicked over the desired object on the map.

To select an object using the Browser:
– The category of object has to be seleceted from the dropdown list of the Data Browser.
– The desired object has to be selected from the list below the category heading.

A6.3.3 *Editing visual objects*

The Property Editor is used to edit the properties of objects that can appear on the Network Map (Junctions, Reservoirs, Tanks, Pipes, Pumps, Valves or Labels). To edit one of these objects, the user should select the object on the map or from the Data Browser, then click the Edit button on the Data Browser (or simply double-click the object on the map). The properties associated with each of these types of objects are described in Tables 1 to 7.

Table 1 Junction properties

Property	Description
Junction ID	A unique label used to identify the junction. It can consist of a combination of up to 15 numerals or characters. It cannot be the same as the ID for any other node. This is a required property.
X-coordinate	The horizontal location of the junction on the map, measured in the map's distance units. If left blank the junction will not appear on the network map.
Y-coordinate	The vertical location of the junction on the map, measured in the map's distance units. If left blank the junction will not appear on the network map. Description: an optional text string that describes other significant information about the junction.
Tag	An optional text string (with no spaces) used to assign the junction to a category, such as a pressure zone.
Elevation	The elevation above some common reference of the junction. This is a required property. Elevation is used only to compute pressure at the junction. It does not affect any other computed quantity.
Base demand	The average or nominal demand for water by the main category of consumer at the junction, as measured in the current flow units. A negative value is used to indicate an external source of flow into the junction. If left blank then demand is assumed to be zero.
Demand pattern	The ID label of the time pattern used to characterize time variation in demand for the main category of consumer at the junction. The pattern provides multipliers that are applied to the Base Demand to determine actual demand in a given time period. If left blank then the Default Time Pattern assigned in the Hydraulic Options **(Report >> Default)** will be used.
Demand categories	Number of different categories of water users defined for the junction. Click the ellipsis button (or press <**Enter**>) to bring up a special Demands Editor which makes it possible to assign base demands and time patterns to multiple categories of users at the junction. Ignore if only a single demand category will suffice.
Emitter coefficient	Discharge coefficient for emitter (sprinkler or nozzle) placed at junction. The coefficient represents the flow (in current flow units) that occurs at a unit pressure drop. It should be left blank if no emitter is present.
Initial quality	Water quality level at the junction at the start of the simulation period. Can be left blank if no water quality analysis is being made or if the level is zero.
Source quality	Quality of any water entering the network at this location. The user should click the ellipsis button (or press <**Enter**>) to bring up the Source Quality Editor.

Table 2 Reservoir properties

Property	Description
Reservoir ID	A unique label used to identify the reservoir. It can consist of a combination of up to 15 numerals or characters. It cannot be the same as the ID for any other node. This is a required property.
X-coordinate	The horizontal location of the reservoir on the map, measured in the map's distance units. If left blank the reservoir will not appear on the network map.
Y-coordinate	The vertical location of the reservoir on the map, measured in the map's distance units. If left blank the reservoir will not appear on the network map.
Description	An optional text string that describes other significant information about the reservoir.
Tag	An optional text string (with no spaces) used to assign the reservoir to a category, such as a pressure zone
Total head	The hydraulic head (elevation + pressure head) of water in the reservoir. This is a required property.
Head pattern	The ID label of a time pattern used to model time variation in the reservoir's head. It should be left blank if none applies. This property is useful if the reservoir represents a tie-in to another system whose pressure varies with time.
Initial quality	Water quality level at the reservoir. Can be left blank if no water quality analysis is being made or if the level is zero.
Source quality	Quality of any water entering the network at this location. Click the ellipsis button (or press <**Enter**>) to bring up the Source Quality Editor.

Table 3 Tank properties

Property	Description
Tank ID	A unique label used to identify the tank. It can consist of a combination of up to 15 numerals or characters. It cannot be the same as the ID for any other node. This is a required property.
X-coordinate	The horizontal location of the tank on the map, measured in the map's unit of scale. If left blank the tank will not appear on the map.
Y-coordinate	The vertical location of the tank on the map, measured in the map's unit of scale. If left blank the tank will not appear on the map.
Description	Optional text string that describes other significant information about the tank.
Tag	Optional text string (with no spaces) used to assign the tank to a category, such as a pressure zone.
Elevation	Elevation above a common reference level of the bottom shell of the tank. This is a required property.
Initial level	Height of the water surface above the bottom elevation of the tank at the start of the simulation. This is a required property.
Minimum level	Minimum height of the water surface above the bottom elevation that will be maintained. The tank will not be allowed to drop below this level. This is a required property.
Maximum level	Maximum height of the water surface above the bottom elevation that will be maintained. The tank will not be allowed to rise above this level. This is a required property.
Diameter	The diameter of the tank. For cylindrical tanks this is the actual diameter. For square or rectangular tanks it can be an equivalent diameter equal to 1.128 times the square root of the cross-sectional area. For tanks whose geometry is described by a curve it can be set to any value. This is a required property.
Minimum volume	The volume of water in the tank when it is at its minimum level. This is an optional property, useful mainly for describing the bottom geometry of non-cylindrical tanks where a full volume versus depth curve will not be supplied.
Volume curve	The ID label of a curve used to describe the relation between tank volume and water level. If no value is supplied then the tank is assumed to be cylindrical.
Mixing model	The type of water quality mixing that occurs within the tank. The choices include: • MIXED (fully mixed), • 2COMP (two compartment mixing), • FIFO (first-in-first-out plug flow), • LIFO (last-in-first-out plug flow).
Mixing fraction	The fraction of the tank's total volume that comprises the inlet-outlet compartment of the two-compartment (2COMP) mixing model. Can be left blank if another type of mixing model is employed.
Reaction coefficient	The bulk reaction coefficient for chemical reactions in the tank. Time units are 1/days. Use a positive value for growth reactions and a negative value for decay. Leave blank if the Global Bulk reaction coefficient specified in the project's Reactions Options applies.
Initial quality	Water quality level in the tank at the start of the simulation. Can be left blank if no water quality analysis is being made or if the level is zero.
Source quality	Quality of any water entering the network at this location. Click the ellipsis button (or press <**Enter**>) to bring up the Source Quality Editor.

Table 4 Pipe properties

Property	Description
Pipe ID	A unique label used to identify the pipe. It can consist of a combination of up to 15 numerals or characters. It cannot be the same as the ID for any other link. This is a required property.
Start node	The ID of the node where the pipe begins. This is a required property.
End node	The ID of the node where the pipe ends. This is a required property.
Description	An optional text string that describes other significant information about the pipe.
Tag	An optional text string (with no spaces) used to assign the pipe to a category, e.g. based on age or material
Length	The actual length of the pipe. This is a required property.
Diameter	The pipe diameter. This is a required property.
Roughness	The roughness coefficient of the pipe. It has no units for Hazen–Williams or Chezy–Manning roughness and has units of mm (or millifeet) for Darcy–Weisbach roughness. This is a required property.
Loss coefficient	Unitless minor loss coefficient associated with bends, fittings, etc. Assumed 0 if left blank.
Initial status	Determines whether the pipe is initially open, closed, or contains a check valve. If a check valve is specified then the flow direction in the pipe will always be from the Start node to the End node.
Bulk coefficient	The bulk reaction coefficient for the pipe. Time units are 1/days. Use a positive value for growth and a negative value for decay. It should be left blank if the Global Bulk reaction coefficient from the project's Reaction Options applies.
Wall coefficient	The wall reaction coefficient for the pipe. Time units are 1/days. A positive value should be used for growth and a negative value for decay. It should be left blank if the Global Wall reaction coefficient from the project's Reactions Options applies.

Table 5 Pump properties

Property	Description
Pump ID	A unique label used to identify the pump. It can consist of a combination of up to 15 numerals or characters. It cannot be the same as the ID for any other link. This is a required property.
Start node	The ID of the node on the suction side of the pump. This is a required property.
End node	The ID of the node on the discharge side of the pump. This is a required property.
Description	An optional text string that describes other significant information about the pump.
Tag	An optional text string (with no spaces) used to assign the pump to a category, e.g. based on age, size or location.
Pump curve	The ID label of the pump curve used to describe the relationship between the head delivered by the pump and the flow through the pump. It should be left blank if the pump is a constant energy pump.
Power	The power supplied by the pump in horsepower (kW). Assumes that the pump supplies the same amount of energy no matter what the flow is. This information is used when pump curve information is not available. It should be left blank if a pump curve is used instead.
Speed	The relative speed setting of the pump (no units). For example, a speed setting of 1.2 implies that the rotational speed of the pump is 20% higher than the normal setting.
Pattern	The ID label of a time pattern used to control the pump's operation. The multipliers of the pattern are equivalent to speed settings. A multiplier of zero implies that the pump will be shut off during the corresponding time period. It should be left blank if not applicable.
Initial status	State of the pump (OPEN or CLOSED) at the start of the simulation period.
Efficiency curve	The ID label of the curve that represents the pump's wire-to-water efficiency (in percent) as a function of flow rate. This information is used only to compute energy usage.
Energy price	The average or nominal price of energy in monetary units per kw-hr. Used only for computing the cost of energy usage.
Price pattern	The ID label of the time pattern used to describe the variation in energy price throughout the day. Each multiplier in the pattern is applied to the pump's energy price.

Table 6 Valve properties

Property	Description
ID Label	A unique label used to identify the valve. It can consist of a combination of up to 15 numerals or characters. It cannot be the same as the ID for any other link. This is a required property.
Start node	The ID of the node on the nominal upstream or inflow side of the valve. (PRVs and PSVs maintain flow in only a single direction.) This is a required property.
End node	The ID of the node on the nominal downstream or discharge side of the valve. This is a required property.
Description	An optional text string that describes other significant information about the valve.
Tag	An optional text string (with no spaces) used to assign the valve to a category, e.g. based on type or location.
Diameter	The valve diameter in inches (mm). This is a required property.
Type	The valve type (PRV, PSV, PBV, FCV, TCV, or GPV). This is a required property.
Setting	A required parameter that describes the valve's operational setting.
	Valve Type Setting Parameter:
	PRV Pressure (mwc or psi)
	PSV Pressure (mwc or psi)
	PBV Pressure (mwc or psi)
	FCV Flow (flow units)
	TCV Loss Coefficient (no units)
	GPV ID of head loss curve
Loss coefficient	Minor loss coefficient that applies when the valve is completely opened (no units). Assumed 0 if left blank.
Fixed status	Valve status at the start of the simulation. If set to OPEN or CLOSED then the control setting of the valve is ignored and the valve behaves as an open or closed link, respectively. If set to NONE, then the valve will behave as intended. A valve's fixed status and its setting can be made to vary throughout a simulation by the use of control statements. If a valve's status was fixed to OPEN or CLOSED, then it can be made active again using a control that assigns a new numerical setting to it.

Table 7 Map label properties

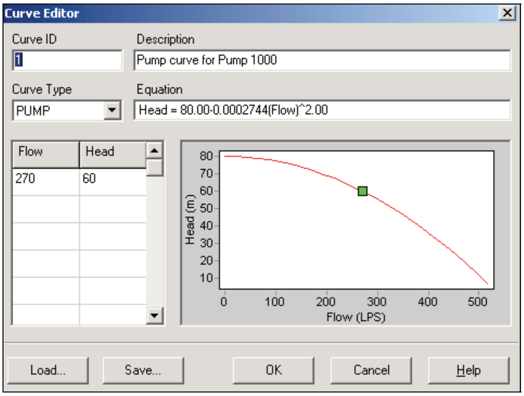
Property	Description
Text	The label's text.
X-coordinate	The horizontal location of the upper left corner of the label on the map, measured in the map's unit of scale. This is a required property.
Y-coordinate	The vertical location of the upper left corner of the label on the map, measured in the map's scaling units. This is a required property.
Anchor node	ID of node that serves as the label's anchor point. It should be left blank if label is not anchored.
Meter type	Type of object being metered by the label. Choices are None, Node, or Link.
Meter ID	ID of the object (Node or Link) being metered.
Font	Launches a Font dialogue that allows selection of the label's font, size, and style.

A6.3.4 *Editing non-visual objects*

Curves, Time Patterns, and Controls have special editors that are used to define their properties. To edit one of these objects, the user should select the object from the Data Browser and then click the Edit button  . In addition, the Property Editor for Junctions contains an ellipsis button in the field for Demand Categories that brings up a special Demand Editor when clicked. Each of these specialized editors is described in the following paragraphs.

Curve editor

The *Curve editor* is a dialogue form as shown in the picture:



The Curve Editor contains the following items:

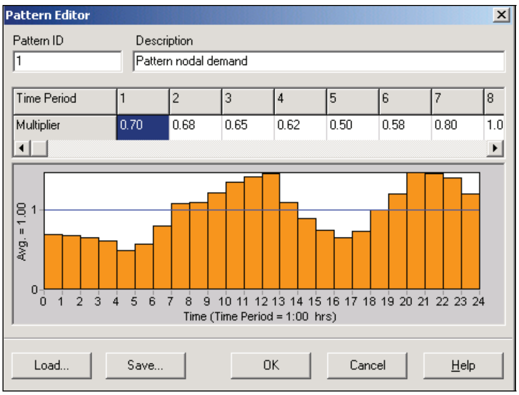
The Curve Editor contains the following items:

Item	Description
Curve ID	ID label of the curve (maximum of 15 numerals or characters)
Description	Optional description of what the curve represents
Curve Type	Type of curve
X-Y Data	X-Y data points for the curve

When moving between cells in the X-Y data table (or after pressing the <**Enter**> key) the curve is redrawn in the preview window. For single- and three-point pump curves, the equation generated for the curve will be displayed in the Equation box. The user can click the **OK** button to accept the curve or the **Cancel** button to cancel the entries. It is also possible to click the **Load** button to load in curve data that was previously saved to file or click the **Save** button to save the current curve's data to a file.

Pattern editor

The *Pattern editor*, when displayed, edits the properties of a time pattern object.



To use the Pattern editor values for the following items should be entered:

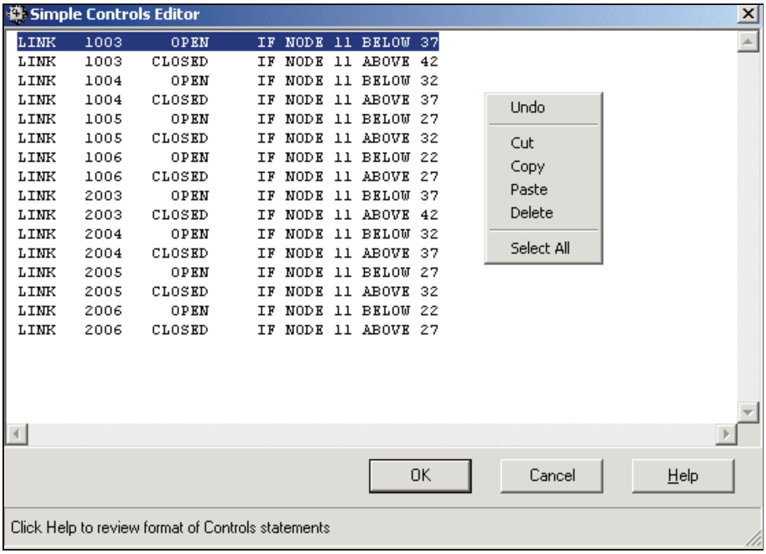
Item	Description
Pattern ID	ID label of the pattern (maximum of 15 numerals or characters).
Description	Optional description of what the pattern represents.
Multipliers	Multiplier value for each time period of the pattern.

Time periods

As multipliers are entered, the preview chart is redrawn to provide a visual depiction of the pattern. If the end of the available *Time periods* is reached when entering multipliers, pressing <**Enter**> adds on another period. When finished editing, clicking **OK** accepts the pattern whilst the **Cancel** button cancels the entries. It is also possible to click **Load** to load in pattern data that was previously saved to file or click **Save** to save the current pattern's data to a file.

Controls editor

The *Controls editor* is a text editor window used to edit both simple and rule-based controls. It has a standard text-editing menu that is activated by right clicking anywhere in the Editor. The menu contains commands for **Undo, Cut, Copy, Paste, Delete**, and **Select All**.

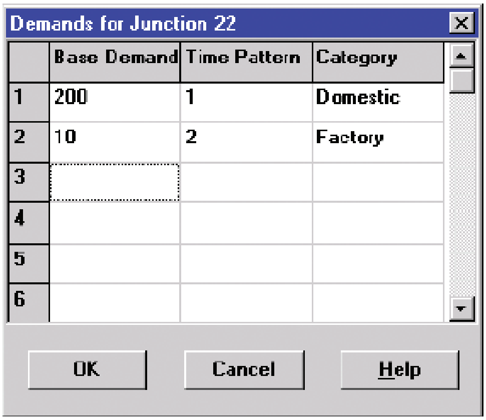


Demand editor

The *Demand editor* is used to assign base demands and time patterns when there is more than one category of water user at a junction. The editor is invoked from the Property editor by clicking the ellipsis button (or pressing <**Enter**>) when the Demand Categories field has the focus. The editor is a table containing three columns. Each category of demand is entered as a new row in the table.

Base demand

The columns contain the following information:
– *Base demand*: baseline or average demand for the category (required)
– *Time Pattern*: ID label of time pattern used to allow demand to vary with time (optional)
– *Category*: text label used to identify the demand category (optional)



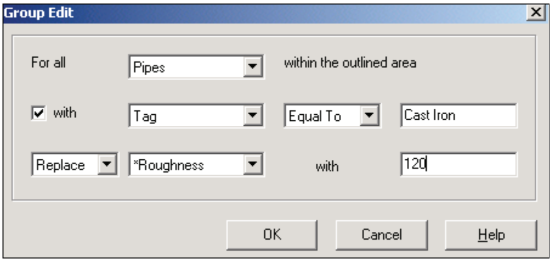
The table is initially sized for 10 rows. Additional rows can be added by pressing <**Enter**> in any cell in the last row. By convention, the demand placed in the first row of the editor will be considered the main category for the junction and will appear in the Base Demand field of the Property Editor.

A6.3.5 *Editing a group of objects*

To edit a property for a group of objects:
– The region of the map that will contain the group of objects to be edited can be selected as follows:
  - Select **Edit** >> **Select Region**.
  - Draw a polygon fence line around the region of interest on the map by clicking the left mouse button at each successive vertex of the polygon.
  - Close the polygon by clicking the right button or by pressing the **Enter** key; cancel the selection by pressing the <**Escape**> key.
  – Select **Edit** >> **Group Edit** from the Menu Bar.
  – Define what to edit in the Group Edit dialogue form that appears.

The Group Edit dialogue form is used to modify a property for a selected group of objects. The steps to use the dialogue form are as follows:
– Select a category of object (Junctions or Pipes) to edit.
– Check the 'with' box if it is necessary to add a filter that will limit the objects selected for editing. Select a property, relation and value that define the filter. An example might be 'with Diameter below 300 (mm)'.
– Select the type of change to make – 'Replace', 'Multiply', or 'Add To'.
– Select the property to change.

– Enter the value that should replace, multiply, or be added to the existing value.
– Click **OK** to execute the group edit.

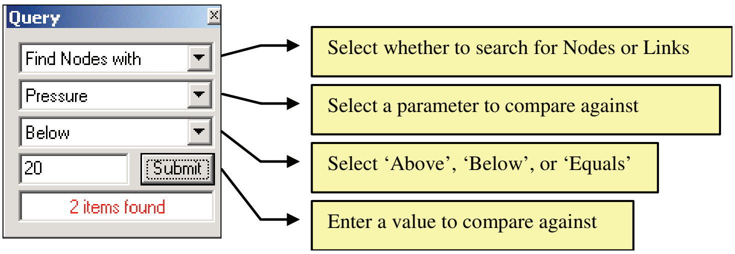


A6.4 VIEWING RESULTS

A6.4.1 *Viewing results on the map*

There are several ways in which database values and results of a simulation can be viewed directly on the Network Map:

- For the current settings on the Map Browser the nodes and links of the map will be coloured according to the colour coding used in the Map Legends. The map's colouring will be updated as a new time period is selected in the Browser.
- ID labels and viewing parameter values can be displayed next to all nodes and/or links by selecting the appropriate options on the Notation page of the Map Options dialogue form.
- The display of results on the network map can be animated either forward or backward in time by using the Animation buttons on the Map Browser. Animation is only available when a node or link viewing parameter is a computed value (e.g. link flow rate can be animated but diameter cannot).
- The map can be printed, copied to the Windows clipboard, or saved as a DXF or WMF file.
- Nodes or links meeting a specific criterion can be identified by submitting a *Map query*; the user should execute the following steps:



Map query

1 Select a time period in which to query the map from the Map Browser.
2 Select **View** >> **Query** or click  on the Map Toolbar.
3 Fill in the following information in the Query dialogue form that appears.
4 Click the **Submit** button. The objects that meet the criterion will be highlighted on the map.
5 As a new time period is selected in the Browser, the query results are automatically updated.
6 It is possible to submit another query using the dialogue box or close it by clicking the button in the upper right corner.

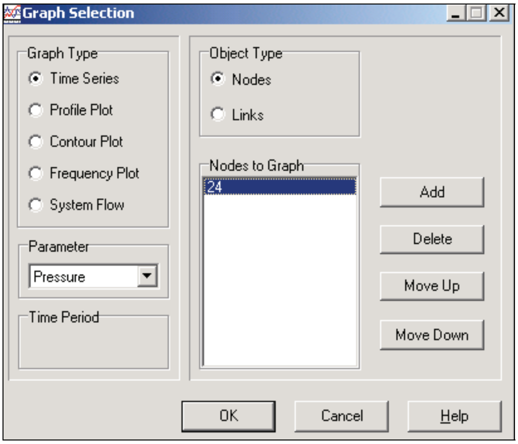
A6.4.2 *Viewing results with a graph*

Analysis results, as well as some design parameters, can be viewed using several different types of graphs. Graphs can be printed, copied to the Windows clipboard, or saved as a data file or Windows metafile. The following types of graphs can be used to view values for a selected parameter:

Type of plot	Description	Applies to
Time Series plot	Plots value versus time	Specific nodes or links over all time periods
Profile plot	Plots value versus distance	A list of nodes at a specific time
Contour plot	Shows regions of the map where values fall within specific intervals	All nodes at a specific time
Frequency plot	Plots value versus fraction of objects at or below the value	All nodes or links at a specific time
System flow	Plots total system production and consumption versus time	Water demand for all nodes over all time periods

The procedure to create a graph is as follows:
– Select **Report** >> **Graph** or click  on the Standard Toolbar.
– Fill in the choices on the Graph Selection dialogue box that appears.
– Click **OK** to create the graph.

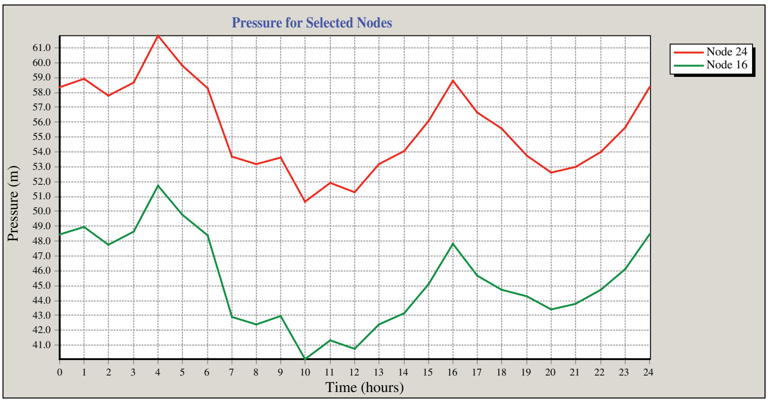
The Graph Selection dialogue is used to select a type of graph and its contents to display. The choices available in the dialogue consist of the following:



Item	Description
Graph type	Selects a graph type
Parameter	Selects a parameter to graph
Time period	Selects a time period to graph (does not apply to Time Series plots)
Object type	Selects either Nodes or Links (only Nodes can be graphed on Profile and Contour plots)
Items to graph	Selects items to graph (applies only to Time Series and Profile plots)

Time Series plots and Profile plots require one or more objects be selected for plotting. The procedure to select items into the Graph Selection dialogue for plotting is as follows:

– Select the object (node or link) either on the Network Map or on the Data Browser. (The Graph Selection dialogue will remain visible during this process).
– Click the **Add** button on the Graph Selection dialogue to add the selected item to the list.



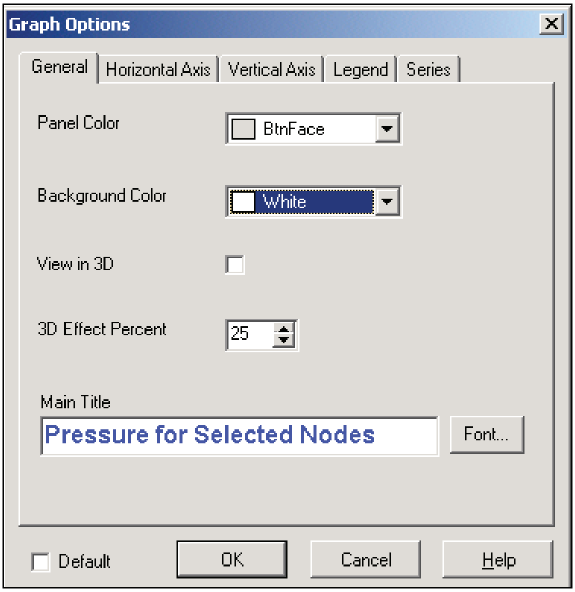
To customize the appearance of a graph, the following steps should be implemented:

– Make the graph the active window (click on its title bar).
– Select **Report** >> **Options**, or click on the Standard Toolbar, or right-click on the graph.
– For a Time Series, Profile, Frequency or System Flow plot, use the resulting Graph Options dialogue to customize the graph's appearance.
– For a Contour plot use the resulting Contour Options dialogue to customize the plot.

The Graph Options dialogue form is used to customize the appearance of an X-Y graph. To use the dialogue box, the user can select from among the five tabbed pages that cover the following categories of options:

– General,
– Horizontal Axis,
– Vertical Axis,
– Legend,
– Series.

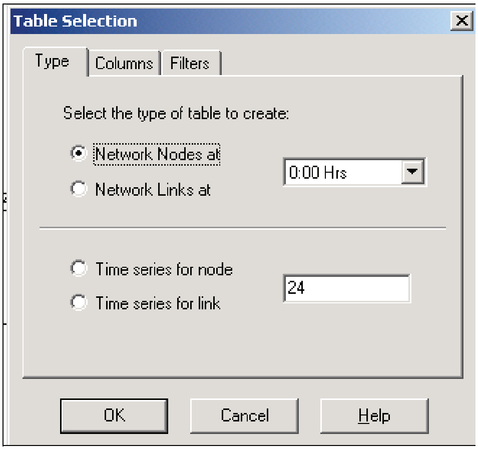
The Default box has to be checked if the current settings are also required as defaults for all new graphs. Clicking **OK** accepts the selections.



A6.4.3 *Viewing results with a table*

EPANET allows selected project data and analysis results to be viewed in a tabular format:

– A Network Table lists properties and results for all nodes or links at a specific period of time.
– A Time Series Table lists properties and results for a specific node or link in all time periods.



Tables can be printed, copied to the Windows clipboard, or saved to file.

To create a table, the user has to:

– Select **View** >> **Table** or click on the Standard Toolbar.
– Use the Table Options dialogue box that appears to select:
  - the type of table
  - the quantities to display in each column
  - any filters to apply to the data

The Table Options dialogue form has three tabbed pages. All three pages are available when a table is first created. After the table is created, only the Columns and Filters tabs will appear. The options available on each page are as follows:

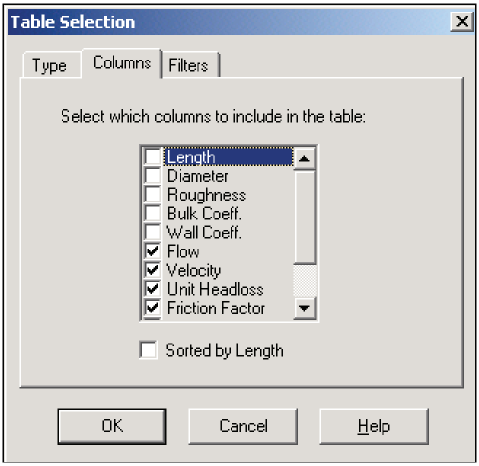
Type page
The Type page of the Table Options dialogue is used to select the type of table to create. The choices are:

- All network nodes at a specific time period.
- All network links at a specific time period.
- All time periods for a specific node.
- All time periods for a specific link.

Data fields are available for selecting the time period or node/link to which the table applies.

Columns page

The Columns page of the Table Options dialogue form selects the parameters that are displayed in the table's columns. The procedure is as follows:

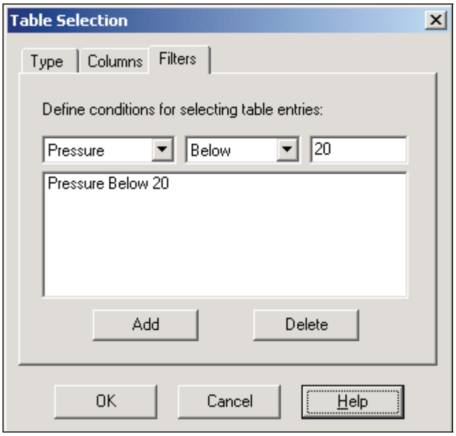


– Click the checkbox next to the name of each parameter to be included in the table, or if the item is already selected, click in the box to deselect it. (The keyboard's Up and Down Arrow keys can be used to move between the parameter names, and the spacebar can be used to select/deselect choices).
– To sort a Network-type table with respect to the values of a particular parameter, select the parameter from the list and check off the **Sorted By** box at the bottom of the form. (The sorted parameter does not have to be selected as one of the columns in the table.) Time Series tables cannot be sorted.

Filters page

The Filters page of the Table Options dialogue form is used to define conditions for selecting items to appear in a table. To filter the contents of a table, the controls at the top of the page should be used to create a condition (e.g. Pressure Below 20). The user can further:
– Click the **Add** button to add the condition to the list.
– Use the **Delete** button to remove a selected condition from the list.

Multiple conditions used to filter the table are connected by ANDs. If a table has been filtered, a re-sizeable panel will appear at the bottom indicating how many items have satisfied the filter conditions.



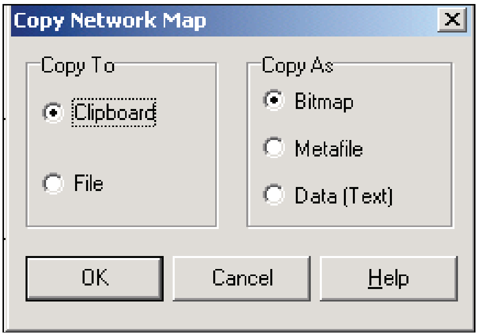
Once a table has been created it is possible to add/delete columns or sort or filter its data. The user has to:

– Select **Report** >> **Options** or click  on the Standard Toolbar or right-click on the table.
– Use the Columns and Filters pages of the Table Selection dialogue form to modify the table.

A6.5 COPYING TO THE CLIPBOARD OR TO A FILE

EPANET can copy the text and graphics of the current window being viewed to both the Windows clipboard and to a file. Views that can be copied in this fashion include the Network Map, graphs, tables, and reports. To copy the current view to the clipboard or to file, the steps are as follows:

– Select **Edit** >> **Copy To** from the main menu or click.
– Select choices from the Copy dialogue that appears.
– and click **OK**.
– If copy-to-file was selected, enter the name of the file in the Save As dialogue box that appears and click **OK**.



Use the Copy dialogue as follows to define how data is to be copied and to where:

– Select a destination for the material being copied (Clipboard or File).
– Select a format to copy in:
  - Bitmap (graphics only)
  - Metafile (graphics only)
  - Data (text, selected cells in a table, or data used to construct a graph)
  - Click **OK** to accept the selections or **Cancel** to cancel the copy request.

A6.6 ERROR AND WARNING MESSAGES

Error number	Description
101	An analysis was terminated due to insufficient memory available.
110	An analysis was terminated because the network hydraulic equations could not be solved. Check for portions of the network not having any physical links back to a tank or reservoir or for unreasonable values for network input data.
200	One or more errors were detected in the input data. The nature of the error will be described by the 200-series error messages listed below.
201	There is a syntax error in a line of the input file created from the network data. This is most likely to have occurred in .INP text created by a user outside of EPANET.
202	An illegal numeric value was assigned to a property.
203	An object refers to undefined node.
204	An object refers to an undefined link.
205	An object refers to an undefined time pattern.
206	An object refers to an undefined curve.
207	An attempt is made to control a check valve. Once a pipe is assigned a Check Valve status with the Property Editor, its status cannot be changed by either simple or rule-based controls.
208	Reference was made to an undefined node. This could occur in a control statement for example.
209	An illegal value was assigned to a node property.
210	Reference was made to an undefined link. This could occur in a control statement for example.
211	An illegal value was assigned to a link property.
212	A source tracing analysis refers to an undefined trace node.
213	An analysis option has an illegal value (an example would be a negative time step value).
214	There are too many characters in a line read from an input file. The lines in the .INP file are limited to 255 characters.

Error number	Description
215	Two or more nodes or links share the same ID label.
216	Energy data were supplied for an undefined pump.
217	Invalid energy data were supplied for a pump.
219	A valve is illegally connected to a reservoir or tank. A PRV, PSV or FCV cannot be directly connected to a reservoir or tank. Use a length of pipe to separate the two.
220	A valve is illegally connected to another valve. PRVs cannot share the same downstream node or be linked in series, PSVs cannot share the same upstream node or be linked in series, and a PSV cannot be directly connected to the downstream node of a PRV.
221	A rule-based control contains a misplaced clause.
223	There are not enough nodes in the network to analyze. A valid network must contain at least one tank/reservoir and one junction node.
224	There is no tank or reservoir in the network.
225	Invalid lower/upper levels were specified for a tank (e.g., the lower lever is higher than the upper level).
226	No pump curve or power rating was supplied for a pump. A pump must either be assigned a curve ID in its Pump Curve property or a power rating in its Power property. If both properties are assigned then the Pump Curve is used.
227	A pump has an invalid pump curve. A valid pump curve must have decreasing head with increasing flow.
230	A curve has non-increasing X-values.
233	A node is not connected to any links.
302	The system cannot open the temporary input file. Make sure that the EPANET Temporary Folder selected has write privileges assigned to it.
303	The system cannot open the status report file. See Error 302.
304	The system cannot open the binary output file. See Error 302
308	Could not save results to file. This can occur if the disk becomes full.
309	Could not write results to report file. This can occur if the disk becomes full.

Warning message	Suggested action
Pump cannot deliver head.	Use a pump with a larger shut-off head.
Pump cannot deliver flow.	Use a pump with a larger flow capacity.
Flow control valve cannot provide additional head at the valve.	Reduce the flow setting on the valve or deliver flow.

A6.7 TROUBLESHOOTING RESULTS

Pumps cannot deliver flow or head

EPANET will issue a warning message when a pump is asked to operate outside the range of its pump curve. If the pump is required to deliver more head than its shut-off head, EPANET will close down the pump. This might lead to portions of the network becoming disconnected i.e. without any source of supply.

Network is disconnected

EPANET classifies a network as being disconnected if there is no way to provide water to all nodes that have demands. This can occur if there is no path of open links between a junction with demand and either a reservoir, a tank, or a junction with a negative demand. If the problem is caused by a closed link EPANET will still compute a hydraulic solution (probably with extremely large negative pressures) and attempt to identify the problem link in its Status Report. If no connecting link(s) exist, EPANET will be unable to solve the hydraulic equations for flows and pressures and will return an Error 110 message where an analysis will be made. Under an extended period simulation it is possible for nodes to become disconnected as links change status over time.

Negative pressures exist

EPANET will issue a warning message when it encounters negative pressures at junctions that have positive demands. This usually indicates that there is some problem with the way the network has been designed or operated. Negative pressures can occur when portions of the network can only receive water through links that have been closed off. In such cases an additional warning message about the network being disconnected is also issued.

System unbalanced

A System unbalanced condition can occur when EPANET cannot converge to a hydraulic solution in some time period within its allowed maximum number of trials. This situation can occur when valves, pumps, or pipelines keep switching their status from one trial to the next as the search for a hydraulic solution proceeds. For example, the pressure limits that control the status of a pump may be set too close together, or the pump's head curve might be too flat causing it to keep shutting on and off.

To eliminate the unbalanced condition it is possible to try to increase the allowed maximum number of trials or loosen the convergence accuracy requirement. Both of these parameters are set with the project's Hydraulic Options. If the unbalanced condition persists, then another hydraulic option, labelled 'If Unbalanced', offers two ways to handle it. One is to terminate the entire analysis once the condition is encountered. The other is to continue seeking a hydraulic solution for another 10 trials with the status of all links frozen to their current values. If convergence is achieved then a warning message is issued about the system possibly being unstable. If convergence is not achieved then a 'System Unbalanced' warning message will be issued. In either case, the analysis will proceed to the next time period.

If an analysis in a given time period ends with the system unbalanced, then the user should recognize that the hydraulic results produced for this time period are inaccurate. Depending on circumstances, such as errors in flows into or out of storage tanks, this might affect the accuracy of results in all future periods as well.

Hydraulic equations unsolvable

Error 110 is issued if at some point in an analysis the set of equations that model flow and energy balance in the network cannot be solved. This can occur when some portion of a system demands water but has no links physically connecting it to any source of water. In such a case EPANET will also issue warning messages about nodes being disconnected. The equations might also be unsolvable if unrealistic numbers were used for certain network properties.

APPENDIX 7

Unit Conversion Table

IMPERIAL (UK) and US ⇒ METRIC[1]	METRIC ⇒ IMPERIAL (UK) and US
LENGTH:	LENGTH:
1 mile = 1.6093 kilometre (km)	1 km = 0.6214 mile
1 yard (yd) = 0.9144 metre (m)	1 m = 1.0936 yd
1 foot (ft) = 0.3048 m	1 m = 3.2808 ft
1 inch (in) = 0.0254 m	1 centimetre (cm) = 0.3937 in
AREA:	AREA:
1 square (sq) mile = 2.5898 km^2	1 km^2 = 0.386 $mile^2$
1 sq yard (yd^2) = 0.8361 m^2	1 m^2 = 1.196 yd^2
1 sq foot (ft^2) = 0.0929 m^2	1 m^2 = 10.764 ft^2
1 acre = 4047 m^2 = 0.4047 hectare (ha)	1 ha = 2.471 acre
VOLUME:	VOLUME:
1 cubic foot (ft^3) = 28.32 litre (l)	1 m^3 = 35.3 ft^3
1 gallon (Imp.) = 4.546 l	1 m^3 = 220 Imperial gallon (gal)
1 gallon (US) = 3.785 l	1 m^3 = 264.2 US gal
WEIGHT/MASS:	WEIGHT/MASS:
1 pound (lb) = 0.4536 kilogram (kg)	1 kg = 2.205 lb
1 ounce (oz) = 28.35 gram (g)	1 kg = 35.27 oz
FLOW:	FLOW:
1 cubic foot per second (ft^3/s) = 28.32 litres per second (l/s)	1 l/s = 0.0353 ft^3/s
1 Imperial mega-gallon per day (mgd (Imp.)) = 52.62 (l/s)	1 l/s = 0.019 mgd (Imp.)
1 US mega-gallon per day (mgd (US)) = 43.81 l/s	1 l/s = 0.0228 mgd (US)
1 Imperial gallon per minute (gpm (Imp.)) = 0.0758 l/s	1 l/s = 13.2 gpm (Imp.)
1 US gallon per minute (gpm (US)) = 0.0631 l/s	1 l/s = 15.85 gpm (US)
PRESSURE:	PRESSURE:
1 pound per square inch (psi) = 6895 Pa (0.06895 bar)	1 bar = 14.5 lb/in^2 (psi)
1 psi = 0.6765 metres of water column (mwc)	1 mwc = 1.478 psi
POWER:	POWER:
1 horse power (hp) = 0.7457 kilo-Watt (kW)	1 kW = 1.341 hp

[1] Some of the units in the table are not metric but are listed because they are in common use.

Temperature scale readings:

$$T_F = \frac{9}{5}T_C + 32\,; \quad T_C = \frac{5}{9}(T_F - 32)$$

The subscripts stand for F (Fahrenheit) and C (Celsius).

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Index

Introduction to Urban Water Distribution comprises the training material used in the Master of Science programme in Municipal Water and Infrastructure at UNESCO-IHE. Participants in this programme are professionals working in the water and sanitation sector from over forty, predominantly developing, countries from all parts of the world. Outside this diverse audience, the most appropriate readers are those who know little or nothing about the subject. However, experts dealing with advanced problems can also use it as a refresher of their knowledge.
The general focus in the contents is on understanding the steady-state hydraulics that forms the basis of hydraulic design and computer modelling applied in water distribution. The main purpose of the workshop problems and the design exercise is to develop a temporal and spatial perception of the main hydraulic parameters in the system for given layout and demand scenarios.
Furthermore, the book contains a detailed discussion on water demand, which is a fundamental element of any network analysis, and general principles of network construction, operation and maintenance. The book includes nearly 500 illustrations and the accompanying CD contains all the spreadsheet applications mentioned in the text and also the network model used in the design exercise.

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