WATER TRANSPORT AND DISTRIBUTION

- PLANNING AND DESIGN OF NETWORK SYSTEMS -

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- PLANNING AND DESIGN OF NETWORK SYSTEMS -

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1 INTRODUCTION

1.1 Description of a water supply system

Potable water is a basic human need. Only 20 or 25 % of the world population has acces to a water supply system, which may vary from a simple well system to an advanced network system.
In Western Europe by the end of the 19th century water supply systems under pressure became available.

The basic aim of a network system is:
- continuity of supply to all consumers of water of sufficient pressure, quantity and quality;
- water supply at acceptable costs.

In a water supply system pressure is essential for reasons of transmission and hygiene. However, many existing water supply systems suffer from pressure problems. These problems may be caused by insufficient design, inferior material quality, unskilled workmanship and lack of maintenance. Lack of pressure may result in an inadequate supply of possible contaminated drinking water. This contamination with ground water, takes place mainly through the pipe joints and is caused by the lack of sufficient "counter" pressure in the water mains.
The potability of the water has to be maintained throughout the entire transport and distribution system. This means that the potable water has to be kept always under pressure understanding and active control of the operation of the water transport and distribution system is therefore extremely important.

In general a water supply system consists of the following components (see also figure 1): source, treatment, storage, transport- and distribution system.

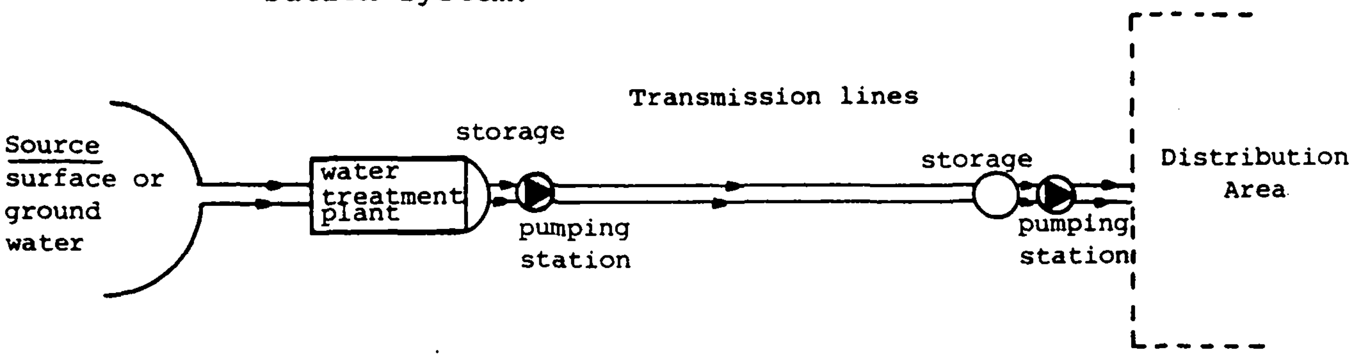


Fig. 1 - Schematic layout of a water supply system

In figure 1 storage has been provided between the treatment plant and the transport system and at the beginning of the distribution system.
Additional storage facilities may be found:
- between source and treatment;
- within the distribution network system;
- with the individual consumer.

(See further paragraph 1.3 and 3.3.)

The necessity of the pumps shown in the above figure depends on the topography. Both transmission and distribution may be by gravity if the natural gradient is sufficient. Also, pumps may be required at the intake or within the treatment plant.

Figure 2 shows the general layout of a distribution system.

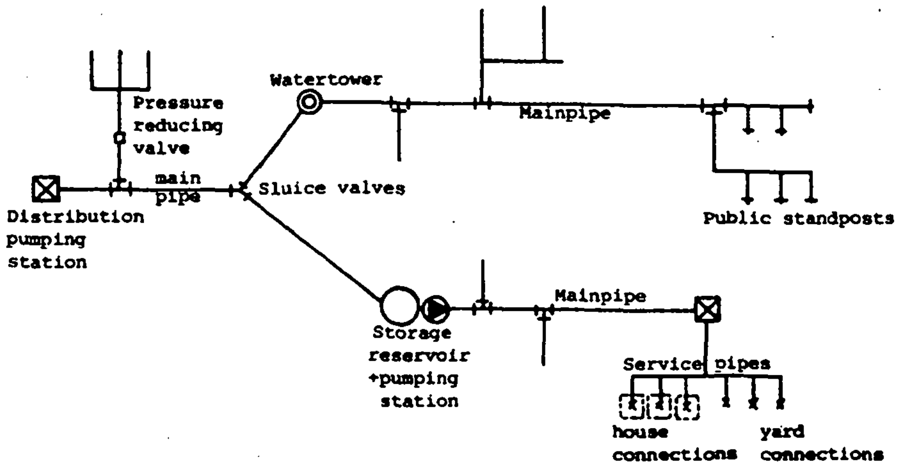


Fig. 2 - Schematic layout of a distribution system

The various components of a water transport and distribution system may be defined as follows.

Transmission line (or trunk main)
Pipeline for the transport of potable, clean water from the water treatment plant to the distribution pumping station or, more in general, to the distribution area.

Main pipe (or distribution main)
Transport of potable, clean water from the distribution pumping station into the various parts of the distribution area.

Service pipe
Pipe from the a main pipe directly to either a public standpost, to a yard connection or to a dwelling.

Public standpost (or standpipe, hydrant or public tap)
A public and communal water point, where a service pipe from the distribution network terminates in one or more taps.

Yard connection
A connection where a service pipe from the distribution network terminates in one or more taps of a private waterpoint within the yard of a dwelling or a (small) number of dwellings.

House connection
A connection where a service pipe from the distribution network terminates at a stopcock of a private domestic installation within a dwelling.

Domestic installation (or plumbing system, water system, drinking water installation))
All piping, tap points and appliances within a dwelling under the responsibility of a private consumer.

For a list of symbols and units see further appendix 1; for a legend for drawings appendix 2.

1.2 Pressure head

As mentioned before pressure is essential for the functioning of a water supply system.
In sanitary engineering it is common use to express water pressure in m.w.c. (meter water column) instead of in the S.I. unit of kPa (1 m.w.c. = 9.8 kPa).
The expression of pressure in m.w.c. enables a very easy comparison of its value with the height of dwellings, hills and elevated tanks.

In general, the pressure applied to the water at the pumping station should be sufficient to maintain the minimum pressure requirement throughout the distribution system (c.f. figure 3).

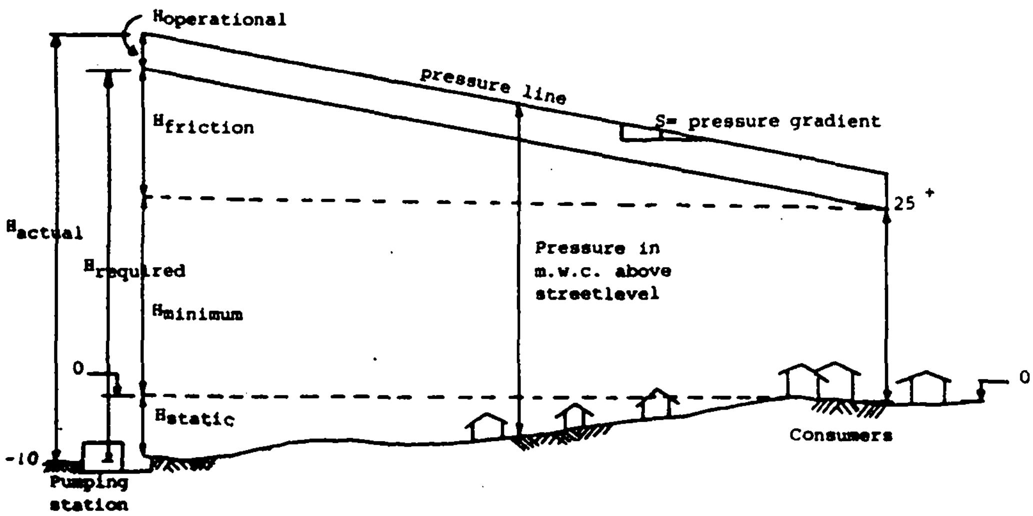


Fig. 3 - The overall pressure head

The following components determine the pressure requirement $H_{required}$ at the pumping station:

a. H_{static}: the pressure representing the difference in elevation between the generally highest point in the distribution system and the pumping station (+ or -).

b. $H_{minimum}$: the required minimum pressure at any point in the distribution system (10-25 m.w.c.).

c. $H_{friction}$: the pressure that is needed to compensate the energy losses between the pumping station and the point that determines H_{static}. This $H_{friction}$ is proportional to the water demand.

d. $H_{required}$: the required pressure at the pumping station. It is the sum of H_{static}, $H_{minimum}$ and $H_{friction}$. This $H_{required}$ is the minimum pressure to be delivered by the pumps to satisfy the pressure required $H_{minimum}$.

e. H_{actual}: the actual operating pressure caused by the pumps. This pressure is determined by the characteristics of the pumps. The pumps need to be selected such that for any given discharge the pressure H_{actual} delivered by the pumps is at least $H_{required}$.

f. $H_{operational}$: the difference between $H_{required}$ and H_{actual}. This $H_{operational}$ signifies the increase of the minimum pressure in the network above $H_{minimum}$.

Some other concepts may be defined as follows:

Pressure gradient (or slope)

The loss of the pressure head pro unit of length. This loss of head can be expressed in m.w.c. per m¹ (m/m), but most of the time it is expressed per km¹ (m/km).

Pressure line

The line which presents the available pressure head at all sequential points of the considered pipeline.

Pipeline- or network-characteristic

The line which shows the pressure requirement at the pumping station for different water demand situations (see also figure 4).
In general the pipeline or network-characteristic is parabolic ($H = f.Q^2$).

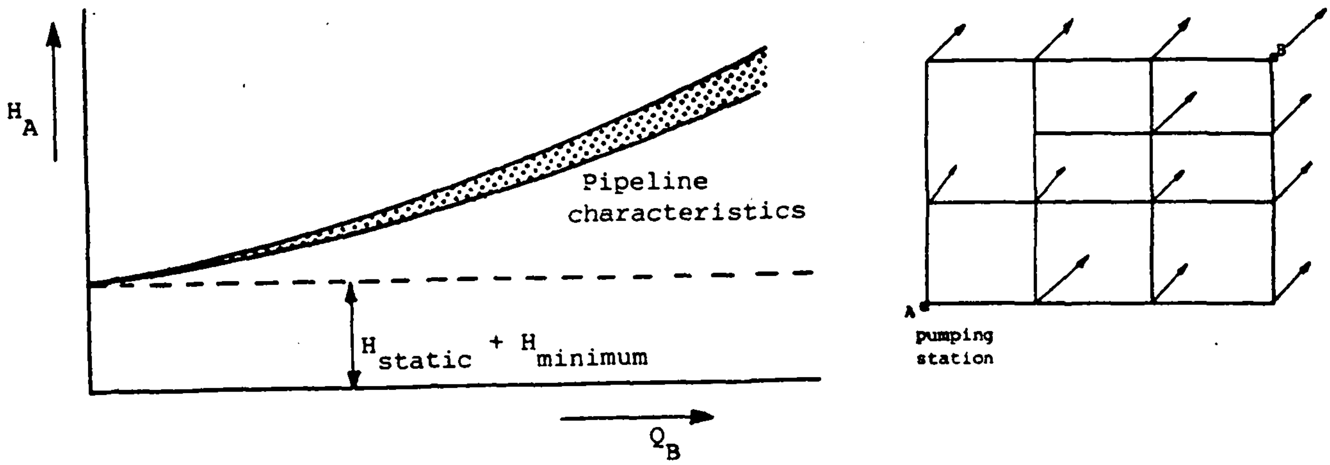


Fig. 4 - Pipeline- or network-characteristic related to point B of a network. Due to the water demand variation at the other nodes there is a small spectrum of characteristics.

Pump-characteristic

The line that represents the pressure head downstreams the pump(-combination) depending on the characteristic of the pump(-combination) (see also figure 5).

The pump characteristic should be supplied by the manufacturer of the pump. In general the pump-characteristic is parabolic ($H = aQ^2 + bQ + c$).

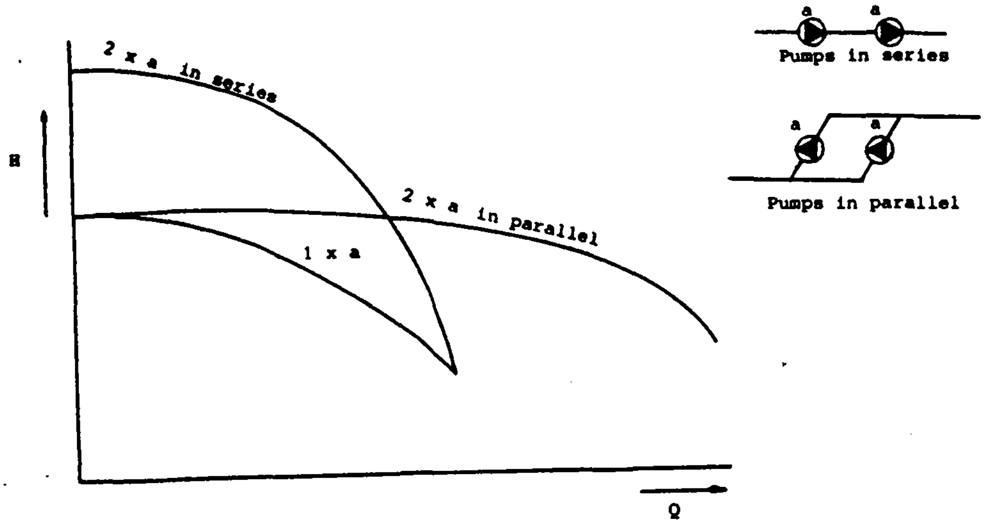


Fig. 5 - Pump-characteristics of pumps in series and in parallel.

When combining pump- and pipeline-characteristics in one figure (see figure 6), the performance of the pump(-combination) can be determined.

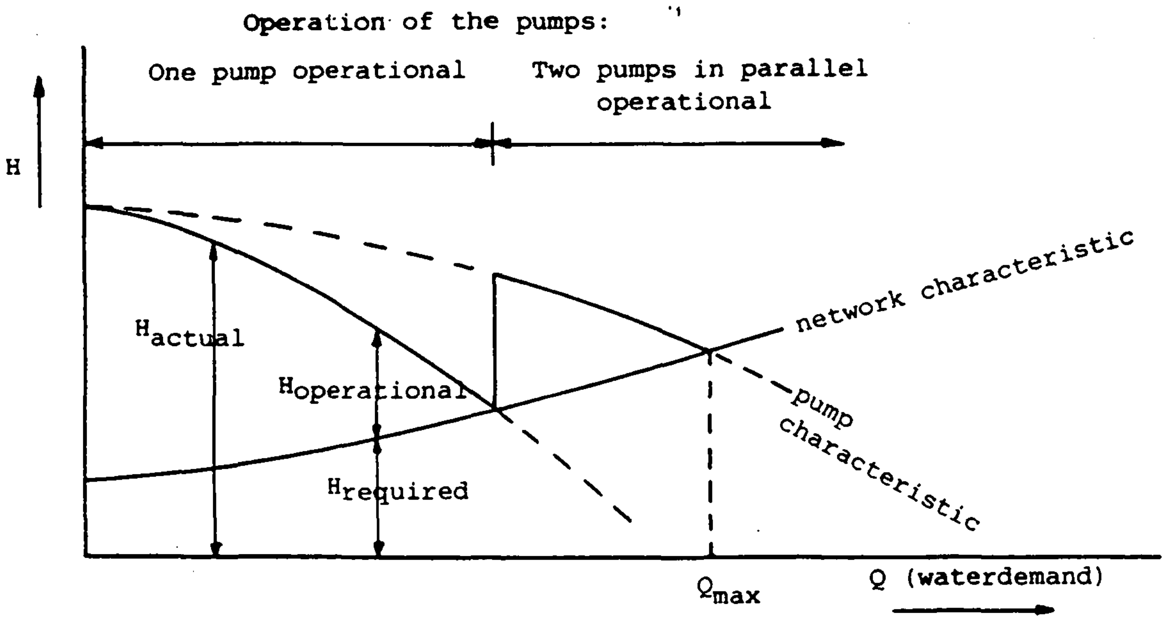


Fig. 6 - The operation of a pumped distribution system

1.3 Storage reservoirs

Storage facilities are part of most distribution systems. The storage reservoir may be at the site of the treatment plant, at the end of the transmission system, or at any other favourable location (e.g. on a hill). Larger distribution systems usually have more than one reservoir. Reservoirs may be constructed as ground tanks or as elevated tanks, the latter being much more expensive per unit volume.

The use of a storage reservoir is to even out differences in incoming supply (from the source or the treatment plant) and outgoing demand (of the consumers). Reservoirs may also serve other, or additional, purposes, e.g. pressure breaking in hilly areas to prevent excessive pressures in the distribution piping.

With regard to the storage facilities at the various locations, the following may be remarked:

a. Storage at the water treatment plant.
   Storage capacity is necessary for operating reasons at the treatment plant. The storage facility evens out the difference between supply (by the treatment works) and demand (by the clear water pumping station). Also it provides a supply of water for backwashing and for internal use. The storage reservoir may also act as a contact basin for chlorination. Usually, the reservoir consists of at least two compartments, so that cleaning can be done whilst maintaining (reduced) storage.

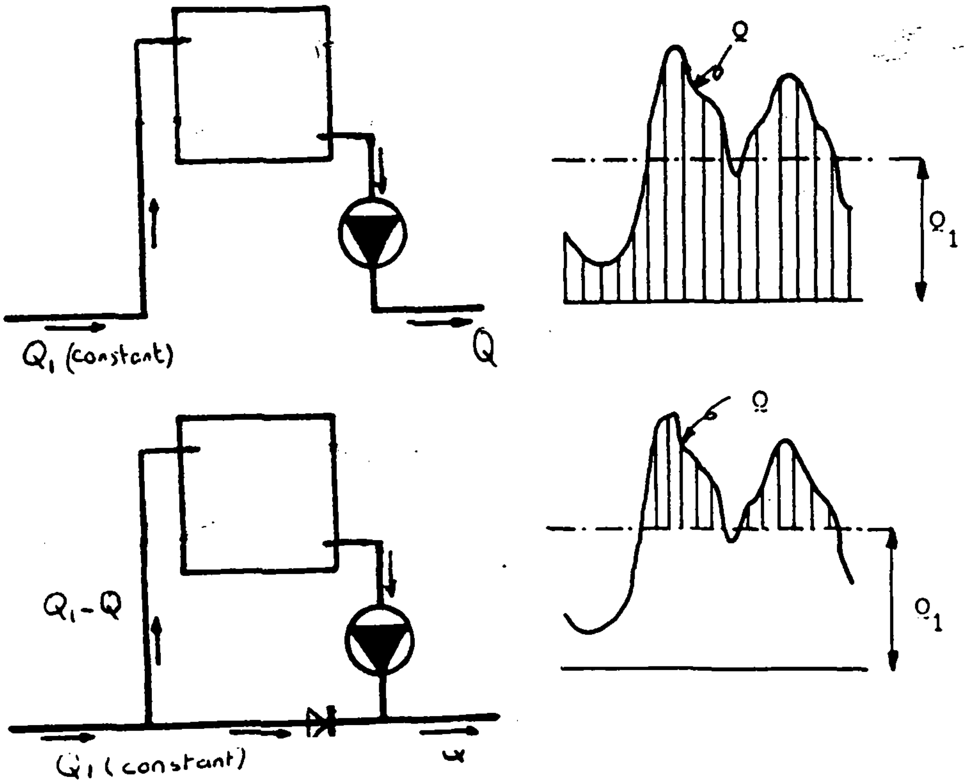
b. Storage at the end of the transmission lines.
Without the use of storage reservoirs at the end of the transmission system, the flow in the entire transmission main must follow consumer demand and will show the same fluctuations. Without using a storage reservoir, the design flow is therefore rather high.
It may therefore be of economic advantage to construct storage reservoir(s) within the distribution system. With the use of storage reservoirs, the fluctuating demand is met from these reservoirs and the flow to the reservoirs is less and more constant.

The cost of the reservoir is easily offset against the lesser cost of the supply main, unless this main is very short. The saving on the supply main is because the diameter of this pipe needs to be sufficient only to convey an average flow, whilst the maximum flow can be supplied drawing the additional requirement from the balancing reservoir.

c. Storage at the consumer's premises.
Consumer reservoirs should be considered in exceptional cases only. For instance:
- for those consumers that would otherwise cause large fluctuations of the water demand in an area;
- for consumers who might otherwise cause potentially dangerous contamination of the distribution water by backflow into the system;
- for consumers who constantly need an uninterrupted water supply.

Note: At house connections and/or public standposts, small reservoirs are sometimes planned, or added at a later stage, when pressure problems arise. Because of the possible danger of contamination, the construction of such private and usually uncontrollable consumer reservoirs must be discouraged.

The way reservoirs may be used within a distribution system is illustrated in figure 7.



All water flows though the reservoir

Only when Q is less than Q_1, water enters the reservoir.

Fig. 7 - The functioning of reservoirs

1.4 Distribution systems

The following types of distribution systems can be distinguisted:
- gravity systems;
- pumped systems;
- combined systems.

Gravity systems

The principal idea of this system is to make use of existing topography. In this way the distribution of potable water can take place without pumping and nevertheless under acceptable pressure (see figure 8).

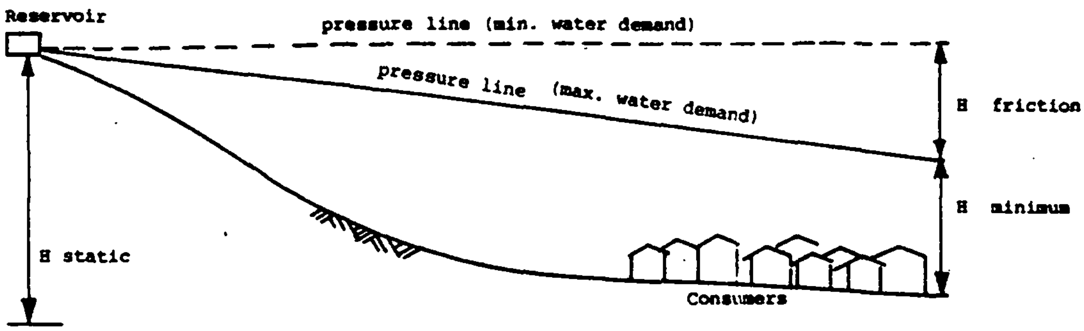


Fig. 8 - A gravity distribution system

Advantages of gravity systems are:
- no energy costs;
- fewer operational problems (fewer mechanical parts, no dependency of electricity supply) and lower maintenance costs.

Disadvantages are:
- less flexibility for future extensions;
- because of the relatively small gradient usually

available for friction losses there may be a need within the whole system for bigger diameters; also the following of contour lines means longer pipelines.

Pumped systems

Pumped water supply systems may be with or without additional reservoirs in the distribution system. When there is no additional reservoir the water supply comes to a standstill in case of a pump failure or power failure (see figure 9).

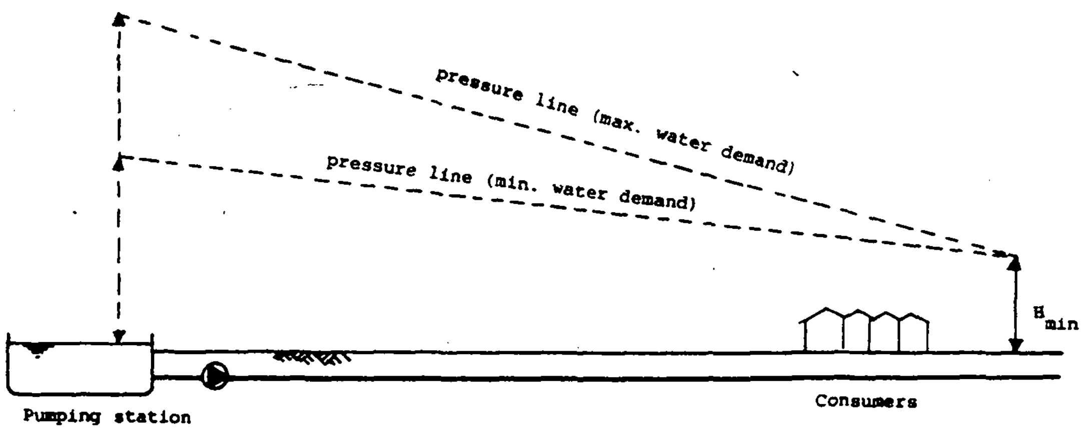


Fig. 9 - A pumped distribution system

In many situations standby pumps and alternative power supply are available.
Also in this type of system pressure variation may be sudden and considerable, especially when the discharge requires a charge-over from one to another pumping stage (see figure 10).

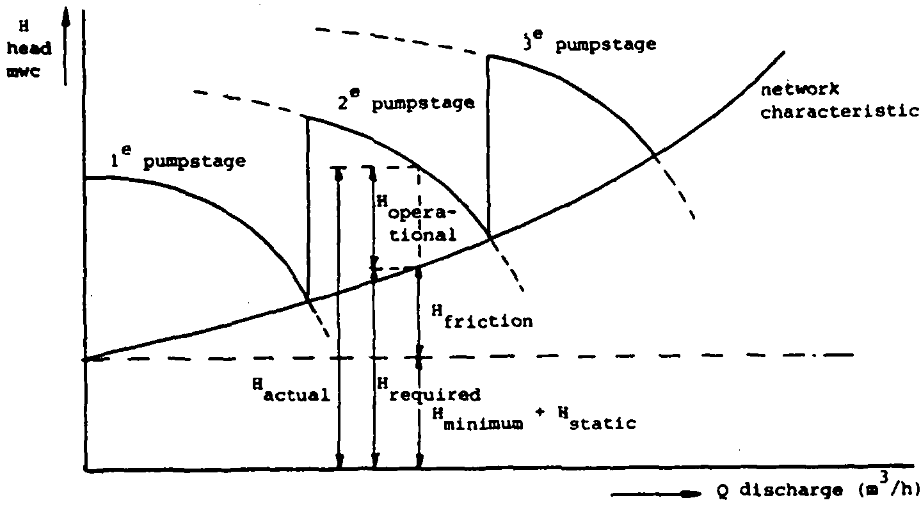


Fig. 10 - Pump- and network-characteristics in a pumped distribution system

Also common is a pumped system with limited storage capacity, (see figure 11) which is usually more reliable. The stored water in an elevated tank serves as a buffer for any accident as a in fire or a power failure.
Also the reservoir controls the pressure in the distribution system.

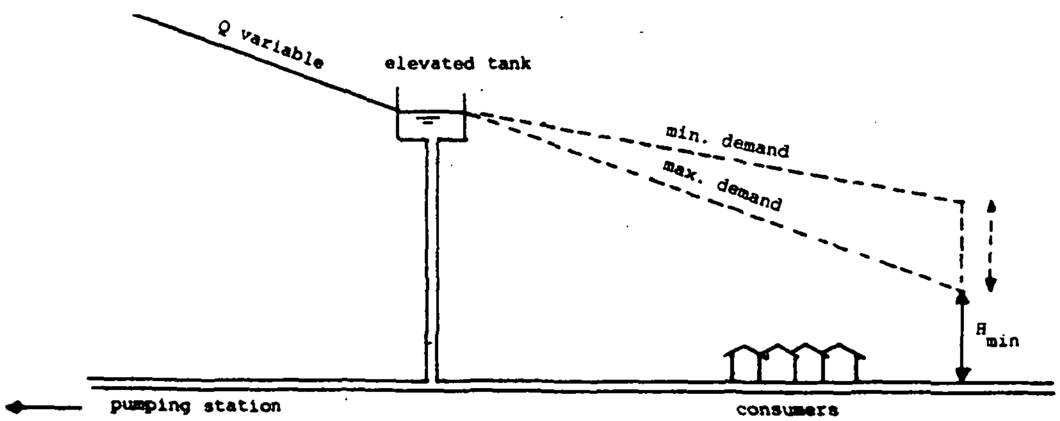


Fig. 11 - The function of an elevated tank in a pumped distribution system

Combined systems

For a combined gravity-pumped system (see figure 11) always storage capacity is necessary to balance in- and outflows. The location of the storage unit(s) is usually determined by the topography.

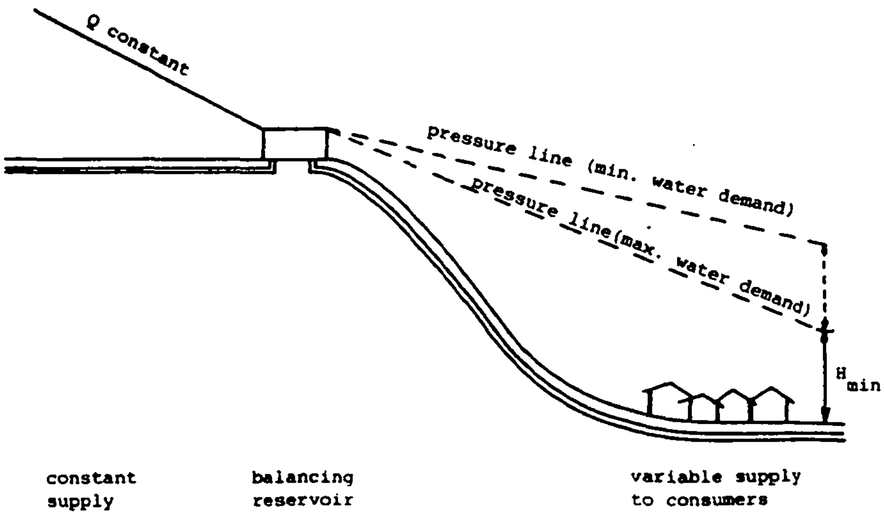


Fig. 12 - A combined pumped and gravity system

Pressure zones

The prevailing topography can also lead to the use of so-called pressure zones (see figure 13). These pressure zones can be formed for economical and technical reasons. By instituting various pressure zones, savings can be obtained in supplying water to the various reservoirs (lower pumping costs) and in the application of lower-class piping due to the lesser pressure. Technically, pressure zones may be advantageous in preventing too high pressures in parts of the network. High pressures are undesirable because of the higher leakage and the increased probability of bursts.

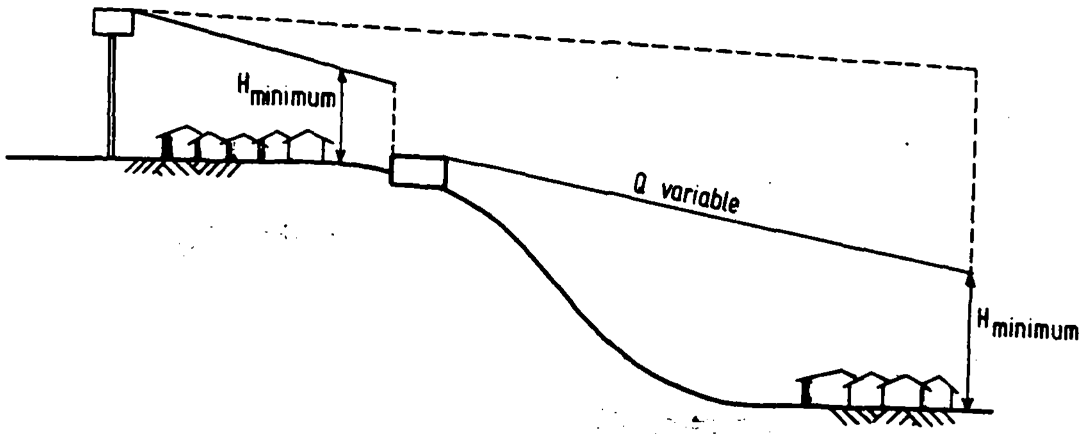


Fig. 13 - Pressure zones

The lower pressure may also be realised by a special appurtenance e.g. a pressure reducing valve (PRV).

2 PLANNING

2.1 Aim of planning

The aim of network systems design is to prepare a lay-out for a distribution system that will:

a. guarantee continuous delivery of a sufficient quantity of safe drinking water to the consumers;
b. be economically and financially viable, ensuring sufficient income for the upkeep and extension of the system.

This aim can be considered as the first priority for the design of the system.

The second priority is that the system

c. have sufficient spare capacity to operate in an emergency situation (power failure, pipe bursts, fires);
d. have a sufficient degree of flexibility with respect to the future.

The capacity of the major components of a distribution system is generally such that the component will perform satisfactory for at least 10 years into the future. It is therefore necessary to know long-term physical planning objectives within the distribution area.

The implementation of a distribution system may be in stages, following actual development of the area. Staged development also allows for adaptation of the design when actual development deviates from the original planning.

2.2 Design period

In most distribution areas water demand is still growing. Excessive growth (e.g. 10 % p.a.) in some areas may be due to a high natural growth rate, together with a large influx of people into the area. In other areas growth is limited and due only to increased water consumption per unit population resulting from rising living standards (introduction of house connections replacing public standposts. Later on also the introduction of special equipment in houses e.g. washing machines, etc.).

In order for a distribution system to operate successfully over a number of years, it is necessary to prepare first a forecast of future water demand. The various components of a distribution system are designed to a capacity which will render them sufficient for a certain period of time. This period is called the design period of the component. During this period, the capacity of the component will be adequate, unless the actual water demand differs from the predictions made (see figure 14).

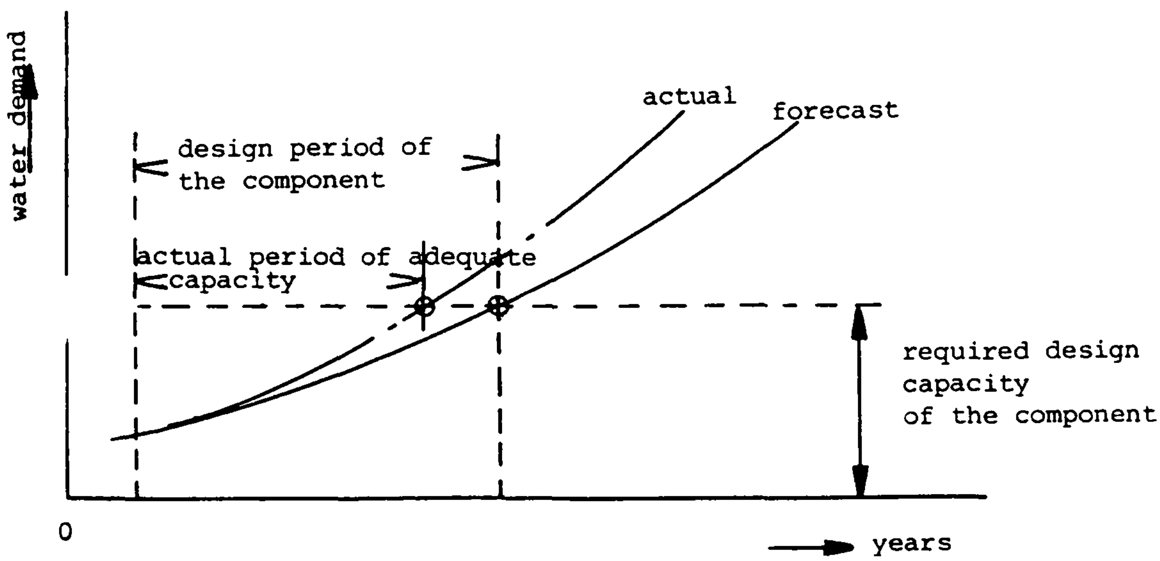


Fig. 14 - Water demand forecast and design period

The technical lifetime of an object represents the period of time it may operate satisfactorily in a technical sense.
The technical lifetime of the various components of a network system is approximately as follows:

a. transmission mains:	30- 50 years
b. reservoirs :	20- 80 years
c. pumping stations	
- structure :	20- 80 years
- equipment :	15- 30 years
d. distribution mains:	30-100 years

The economical lifetime of an object represents the period of time it can operate without being more costly than its replacement. The economical lifetime is never longer but usually shorter than the technical one.
The estimation of the economical lifetime is complex. It depends on aspects such as operation and maintenance costs, technological advancement (e.g. energy-saving) and interest rates, but not on the used method of depreciation.
In practice the economical lifetime is quite often used as the design period. Furthermore, to simplify the design, the design periods of certain components are equalized.
The following are typical design periods:

- treatment plants	15-25 years
- reservoirs	15-25 years
- pumping stations excl. pumps	15-25 years
- pumps	10-15 years
- transmission lines and distribution networks	15-25 years.

The selection of the most appropriate design period may be done using the present value method (c.f.

appendix 3).
Experience has shown that design periods are rarely longer than 25-30 years and seldom shorter than 5-10 years. Extremely short design periods are also undesirable from the practical point of view.

The execution of a design based on a design period of, say, 25 years, may be in stages. In this way the extension of a distribution system follows the actual development in the distribution area, whereby the diameter of the pipes being laid follows from the hydraulic calculations made for the end of the design period. Executing a plan this way also provides the opportunity to evaluate and possibly review the plan.

3 WATER DEMAND

3.1 Water consumption

Water consumption is usually distinguished in a domestic and a non-domestic component. Apart from consumption, every system is subject to leakage. Water consumption and leakage together make up the demand for potable water. This demand is not constant, but varies hourly, daily, and generally also seasonally.

a. Domestic water consumption

Domestic water consumption is the product of the number of the population served and the unit domestic water consumption.

The number of population served is the product of the total population in the service area and the factor expressing the percentage of the population with access to the distribution system (also called coverage).

The unit domestic water consumption is the quantity of water for domestic use withdrawn from the network by an individual consumer. Unit domestic consumption is generally expressed in litres per capita per day.

A number of factors affect the unit domestic water consumption:

- income
- socio-cultural habits
- the type of water connection, i.e. a public standpost, a yard connection or a house connection
- the characteristics of the water in terms of quality, quantity and price
- the availability of alternative water resources.

For a projection of domestic water consumption, data collected in the existing situation should be evaluated considering the above factors.

The most important factors determining future water demand are: percentage coverage of the population by the water supply system, population growth and type of service connection.
Percentage coverage may be nil to very low in some less developed areas and policy goals need to be established to increase the coverage to almost 100 % over an acceptable period of time.

Unit consumption rates from the three types of service connections mentioned may vary with a factor 4 to 5 (see Table 1) and distribution of the served population over these three groups has to be assumed for the future in order to predict water demand. A gradual change is normally assumed from standpipes to house connections.

The World Health Organisation distinguishes three stages as goals

First stage : 90 % of the served population draws water from public standposts and 10 % has house or yard connections.

Second stage: 50 % of the served population is supplied by means of public standposts and the remaining 50 % has house or yard connections.

Third stage : the entire population is supplied by means of house connections.

At this moment most rural water supply systems in lesser developed countries, because of the limited financial resources, serve the majority of their consumers, through public standposts.
Unit consumption from public standposts is strongly subject to the maximum each household cares to carry home from the nearest standpost.
In that respect the selection of the average walking distance (up to 200 metres) plays an important role.
The average demand at public standposts may vary between 20-60 l/cap/day and with yard and house connections between 100-300 l/cap/day.
A WHO statistical report gives the following daily domestic consumption in urban and rural areas (table 1).

Water consumption l/cap/day	URBAN AREAS				RURAL AREAS	
	House connections		Public standposts			
	min.	max.	min.	max.	min.	max.
Africa	65	290	20	45	15	35
Central & S. America	160	390	25	50	70	190
Eastern Mediterranean	95	245	30	60	40	85
North Africa & Turkey	65	210	25	40	20	65
Southern Asia	75	165	25	50	30	70
Western Pacific	85	365	30	55	30	95
Average	90	280	25	55	35	90

Table 1 - Daily domestic waterconsumption in urban and rural areas (WHO 1973)

b. Non-domestic water consumption

Non-domestic water consumption includes industrial water consumption, commercial water consumption and water consumption at schools, hospitals, and public buildings. Non-domestic water consumption may be from 20 % to more than 100 % of domestic water consumption. Consumption depends mainly on the degree of industrialization of the service area. Water requirements by industry differ with the type of industry. Bottling plants, breweries, the canning industry and abattoirs are known to be large water consumers (see table 2). So are the steel, paper, textile and chemical industries. However, due to the large unit size of these enterprises, they usually have their own water supply arrangements.

Careful investigation into future industrial developments is essential for the accurate prediction of non-domestic water consumption.

Industry	Water demand
Abattoirs	5- 30 m^3/ton carcass
Bottle washing plant	2- 6 l/bottle
Breweries	20- 30 m^3/ton of malt
Canning plants	5- 70 m^3/ton product
Dairy industry	2- 17 m^3/ton product
Chemical industry	200-1000 m^3/ton product
Paper manufacturing	50- 500 m^3/ton product
Steel industry	5- 400 m^3/ton product
Textile industry	15-1000 m^3/ton product

Table 2 - Unit water consumption for industries

Industrial demand can also be related to the number of employees or to the size of the industrial area.
Expressed in l/day per employee for manufacturing industries for instance there can be an average daily water demand of 100-1500 l/day per employee and for the construction sector of 50-200 l/day per employee.
Related to the size the industrial area the required water demand will be expressed in m^3/ha (f.i. 10-30 m^3/ha).

It should be noted that the peak in industrial water consumption does not necessarily concur with the peak in domestic water consumption.

A special type of non-domestic water consumption is formed by the irrigational water demand (agriculture fields, gardening, public parks and recreational facilities).
Normally, this kind of water consumption is not a separate category. In cases, however, of extremely high irrigational demand, the requirement may be calculated separately. This is especially the case where the use of an alternative source specifically for irrigation needs to be evaluated.

c. Leakage

From examinations and wide experience, it is known that quantities of water are lost due to leakage and wastage in the distribution system and in the domestic installations. In preparing the water consumption projection leakage, unfortunately, is therefore one of the basical design criteria.
It should be realized that all water lost as

leakage and wastage has to be produced, pumped, transported and distributed without generating any income to the water supply organization. The leakage percentage may vary between 10 % and 60 % (or even more!) of the total supply depending on local soil conditions, and on the age and the operation and maintenance of the distribution system. (See WATER TRANSPORT AND DISTRIBUTION - Part "Operation and Maintenance of Network Systems.")

3.2 Fluctuation of water demand

Each individual component of a water supply system should be designed such that it can meet the design criteria under the maximum flow conditions to be expected at that element at the end of the design period.
The various components of a system may or may not be subject to fluctuations in water demand. The degree to which a component is subject to demand fluctuation depends on the location of the element in the water supply system. Commonly, demand fluctuations are distinguished as being seasonal, monthly, weekly, daily, hourly, or instantaneously. A certain factor is generally used in the design of distribution systems expressing the quotient of the flow prevailing at the time and the average flow conditions. In this way different factors may be defined expressing daily demand to average daily demand, hourly demand to average hourly demand, etc..
In general the multiplication of the maximum day factor and the maximum hour factor is called: peak factor (p.f.).

Water consumption pattern

To get an impression of the actual fluctuations of the water consumption, measurements are required. Usually this kind of measurements is not available completely. Measurement of at least the maximum water consumption is recommended. But even when all kind of measurements are done a further schematization is necessary for design purposes.
Some examples of demand patterns are presented in figure 15.

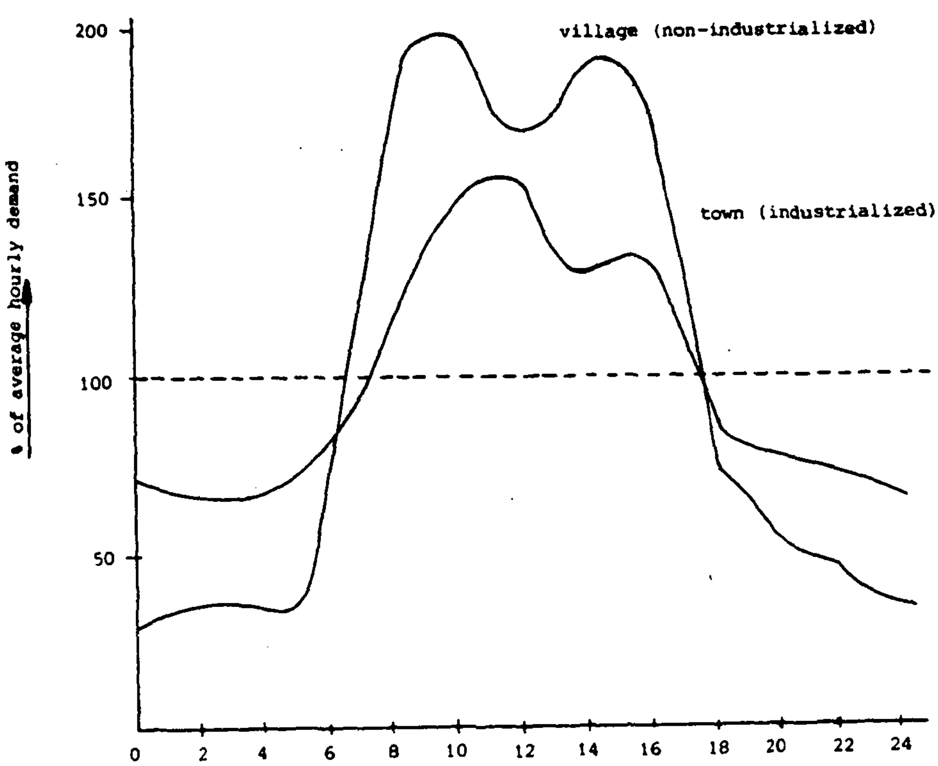


Fig. 15 - Actual water consumption patterns of a town and a village

An example of the fluctuation of the water demand, is shown in table 3. The hourly factor is defined as the demand during the maximum hour on a maximum day divided by the average hour of an average day. In table 3 this factor is 1.8 and 2.5 respectively

for the two villages.
The hourly factor can be rather high in rural areas, without (a moreless constant) industrial water demand and with uncontrolled irrigation activities. A high minimum hourly demand, as shown for the first village in table 3 may be an indication of excessive leakage in the network.

Water demand	Village with some industrial use High leakage Low pressure		Village without industrial use Some leakage Normal pressure	
	max.	min.	max.	min.
Yearly demand	100 %	100 %	100 %	100 %
- dry seazon	105 %	99 %	102 %	98 %
- wet seazon	96 %	92 %	99 %	90 %
Monthly	115 %	90 %	120 %	85 %
Weekly	120 %	85 %	125 %	80 %
Daily	135 %	80 %	150 %	70 %
Hourly	180 %	40 %	250 %	15 %
Momentaneously	>180 %	40 %	>250 %	15 %

Table 3 - Water demand variation expressed in % of the average hour of a year

The different factors can be low when:
- there is industrial activity. Most industries have a constant water demand during operational hours;
- there is excessive leakage or permanent waste in the distribution network;
- there is a water demand in excess of the

available capacity. This may cause a substantial water demand during the night.

The required factors can be estimated by collecting data from the system itself or by using these of simular villages or towns in the region.
It is not recommended to use figures from systems operating under total different circumstances.

The mentioned factors may be considerably reduced by promoting specific consumer storage. In this respect some remarks should be made:

a. Reduction of peak flow to industrial areas may be achieved by offering water at a reduced tariff during off-peak hours.
b. The required presence of individual storage reservoirs as a policy may be recommended for industrial consumers.
c. This system is not often recommended for house connections because of hygienic reasons. Besides there are costs implications: Although the costs of individual storage must often be met by the individual consumers, and not by the water authority, the accumulated construction costs to the community are usually higher than in the case of centralized storage.

The hydraulic design of the main elements of a distribution network is based on the projected maximum hourly demand. Within smaller parts of the (e.g. within a small village or within town quarters) the instantaneous demand can be considerably higher than the calculated peak hourly demand. Thus for calculations of water mains in small sub-areas the hourly factor is not used. Instead, the so-called simultaneity factor is applied. The simultaneity factor expresses the relationship between instantaneous and average demand, and decreases with an increasing number of consumers to equal the peak factor generally between 1000 and 5000 consumers (see also figure 16).

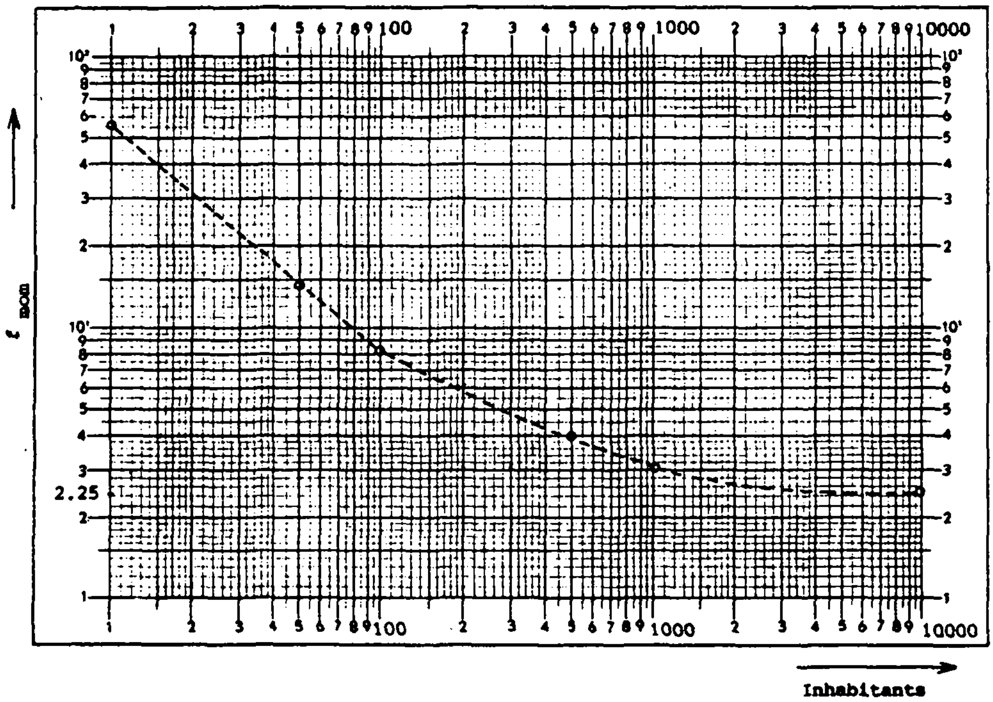


Fig. 16 - An example of the simultaneity factor.

The decrease of the simultaneity factor may be illustrated by the following example. Say the number of inhabitants per connection averages 4 and the average daily demand is 450 litres per connection. If now within one single house connection a bath mixing faucet with a capacity of 25 litres per minute is opened, this represents a demand of 36000 litres on a daily basis, or 80 times the average daily demand. The simultaneity factor in this case is 80.

Now consider two house connections: the probability that both bath mixing faucets are running simultaneously is rather limited. The maximum demand on a daily basis for two house connections is therefore not much larger than 36000 litres. Consequently, the simultaneity factor for two house connections (8 people) is not much larger than 40 - say about 50. Likewise, the simultaneity factor can be calculated to decrease for larger number of consumers.

3.3 The capacity of storage facilities

Reservoirs may be located at the treatment plant, at the end of the transmission system and in the distribution system. Additionally, there may be consumer owned reservoirs located at their premises (see paragraph 1.3).
The main aim of a storage reservoir is to even out differences between incoming supply and outgoing demand. In this way water supply from the reservoir at the treatment plant can continue wether or not the treatment plant is operational at that time. Likewise, the storage reservoirs at the end of the transmission line or in the distribution system can supply the ever fluctuating water demand, whilst being supplied with a more or less constant flow of water from a transmission line or a main pipe. Also consumer owned reservoir usually serve to even out the difference between constant supply by the water company and fluctuating consumer demand.
Reservoirs are financially justified when their costs are ofset against the lesser costs of the supply main, that can be reduced in diameter when designed to supply average instead of peak demands.

The capacity of storage reservoirs depends on the characteristics of the system and, more especially, on the distance between treatment plant and distribution area, and on the variation of supply and demand. The minimum storage requirement in the distribution area following from the latter increase with the distance between treatment plant and distribution area.

The reservoir volume required to equalize supply and demand can be calculated if the fluctuation of supply and demand is known. Table 4 and figure 16 show a sample calculation.

(1)	(2)	(3)	(4)	(5)	(6)
				(4)-(3)	Σ(5)
Time span	water demand expressed as % of average hourly demand	water demand	water production	production less demand	fluctuation of storage
		expressed as a percentage of daily demand			
0-3 hr	25	3	12,5	+ 9,5	17,5
3-6	35	4	12,5	+ 8,5	26
6-9	130	16	12,5	- 3,5	22,5
9-12	200	25	12,5	- 12,5	10
12-15	160	20	12,5	- 7,5	2,5
15-18	120	15	12,5	- 2,5	0
18-21	85	11	12,5	+ 1,5	1,5
21-24	45	6	12,5	+ 6,5	8
		100	100		

Note: when sign in column 5 is positive, excess production goes to storage; when this is negative, excess demand is supplied from storage.

Table 4 - Sample calculation of required storage volume. (The situation at the starting moment is that there is 8 % storage allready!).

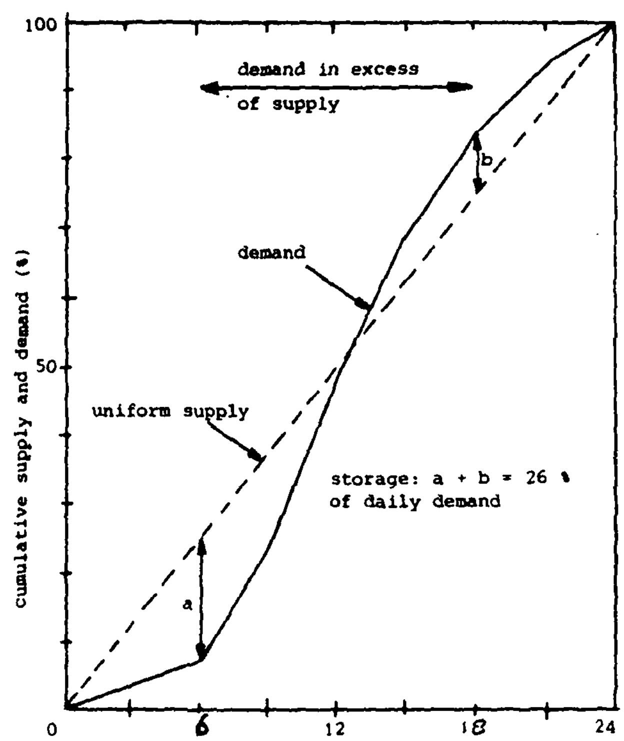


Figure 16 - Sample graphical calculation of required storage volume

Procedure:

1. Draw a cumulative demand curve.
2. Draw a cumulative supply curve (here uniform supply 24 hr/day).
3. Scale the maximum ordinates a and b between supply and demand lines.
4. Calculate the required storage as sum of a and b
5. Choose the exact storage volume to be constructed on base of an evaluation of the pregoing steps (accuracy, future developments, reliability)

In case of uniform supply, i.e. with the treatment plant operational 24 hours a day, the typical storage requirement to even out differences between supply and demand is 25 to 35 % of the average daily demand. In case of longer transmission lines (> 20 km) this storage volume is constructed at the end of the transmission main. Some additional storage is constructed at the treatment plant. This storage volume of 5 to 10 % of the average daily demand, overcomes temporary interruptions of production and internal water requirements.

With increasing length of the pipeline the most economical storage requirement may increase up to 150 % of the average daily demand.
With very short transmission lines there is usually no storage in the distribution area and all storage is constructed at the treatment plant. This simplifies operation and maintenance. The typical total storage requirement in this case is 35 % to 40 % of the average daily demand. Table 5 summarizes the storage requirements.

length of the transmission line (km)	required volume of stored water as a percentage of the average daily demand	
	at the treatment plant	in the distribution area
< 20	35 to 40	--
20 - 50	5 to 10	25 to 35
50 - 100	5 to 10	75
> 100	5 to 10	100 to 150

Table 5 - Required volume of stored water in relation to the length of the transmission line

4 DESIGN CRITERIA

4.1 Minimum and maximum pressure

The minimum pressure (H_{min}) is required in networks for hygienic reasons. In general 5 m.w.c. above the highest tap is sufficient. For urban areas this means H_{min} ≈ 15-20 m.w.c. above ground level. For rural areas where a public standpost is the highest tap an H_{min} of ≈ 6 m.w.c. above ground level may suffice.
In case there are fire fighting requirements (for instance a flow of 50 m^3/h at a pressure loss of 10 m.w.c.) the required minimum pressure will be at least 10 m.w.c. above ground level.

The maximum pressure in the network will be approximately 60 to 80 m.w.c.. Material properties of the pipes are taken into account.
Pressure reducing valves may be used if necessary.

4.2 Design pressure gradients

The loss of pressure head per unit length, the pressure gradient, depends on the overall available pressure head and the length over which the water need to be transported. This indicates a socalled "permissable" pressure gradient to be used for design purposes. This permissable pressure gradient depends on the adopted minimum and maximum pressures within the network, local topographic circumstances, and the (future) size of the network.

Table 6 (among many others) should be used very carefully and not as a directive. It shows a practical range of pressure gradients for various pipeline diameters, as derived from practical experience.

Diameter [mm]	Design pressure gradient [m/km]	Design velocity [m/sec]
100	4 - 6	≈ 0.5
125	3 - 4	≈ 0.5
150	2 - 3	≈ 0.5
200	1.5 - 2.5	≈ 0.5
250	1 - 2	0.5-1.0
300	1 - 1.5	0.5-1.0
≥ 400	0.5 - 1.0	0.5-1.5

Table 6 - Practical range of design pressure gradients or design velocities

4.3 Design velocity

Instead of the pressure gradient the design velocity can be used as a design criterium.
Depending on the diameter, the design velocity is usually in the range of 0.5 - 1.5 m/sec (see table 6, but also appendix 7).

5 ECONOMIC ANALYSIS

5.1 Costs

When designing a new or the extension of an existing, transport and distribution system, there is usually a number of technically acceptable solutions. These solutions may be compared on the basis of reliability of supply, flexibility with regards to future extensions, and on the basis of cost.

When comparing on the basis of cost, the objective is to find an alternative that conveys the water at minimum cost. This means that not only the construction capital costs have to taken into account, but also the operation and maintenance recurrent costs.

Several methods are available to evaluate capital and recurrent costs. The present value method is often used in the economic evaluation of alternative solutions for a water transport and distribution system (see also Appendix 3). This method, by using a discount rate, calculates all future costs back to one reference year and adds the discounted costs to a total called the present value. The alternative with the lowest present value is then recommended as being the most economical solution.

The capital costs may be expressed as:

$$I = a + b.D^{c}$$

where I = investment costs per metre pipe
D = diameter of the pipe in m
a, b, c = constants to be determined from previous projects

The <u>operating (energy) costs</u> may be expressed as:

$$k_i = \frac{u_i * Q_{equiv.i} * \Delta H_i * e_i}{367 * \eta_{equiv.i.}}$$

where

k_i = energy costs in year i

u_i = number of pumping hours (h) in year i

$Q_{equiv.i}$ = equivalent constant flowrate during pumping (m^3/h) in year i

ΔH_i = head loss (m) when pumping $Q_{equiv.}$ in year i

e_i = enerqy costs per kWh in year i

$\eta_{equiv.i}$ = equivalent efficiency of energy supply in year i.

367 = conversion factor

It should be noted that when considering values for $Q_{equiv.}$ and ΔH, these values should be larger than the average flow rate $Q_{av.}$ and average pressure loss ΔH_{av}, as energy costs increase exponentially with the third power of Q, meaning that the pumping o f flow rates in excess of $Q_{av.}$ cost relatively more than is saved when pumping less than $Q_{av.}$. A value for Q equivalent to 1.3 x $Q_{av.}$ may be used ($Q_{equiv.}$).

The value for $\eta_{equiv.}$ should take into consideration the efficiency of the power supply system (efficiency of motor and pump, power factor), and also the head $H_{operational}$ delivered by the pump at a certain flow rate, when head may not be equal to the head required by the system (system characteristic) $H_{friction}$. (See also paragraph 1.2.)

The value for $\eta_{equiv.i}$ =

$$\frac{H_{friction}}{H_{operational} + H_{friction}} \times \eta$$

Values for $\eta_{equiv.i}$ may be as low as 0.2 to 0.3.

The maintenance costs may be calculated as:

$$k_m = C_m \cdot I,$$

where

k_m = annual maintenance costs
C_m = factor expressing the maintenance costs as a percentage of the investment costs
I = investment costs.

The value of C_m may be:

- for networks : 0.01 to 0.015;
- for reservoirs : 0.005;
- for pumping stations: 0.02 to 0.03.

After calculation of the above costs for the economic lifetime of the pipeline (say about 15 to 25 years) and applying the present value method, the costs per m³ water conveyed may be calculated for each alternative considered.
Figure 17 shows the costs per m³ water conveyed for a transmission line for which various diameters were considered.

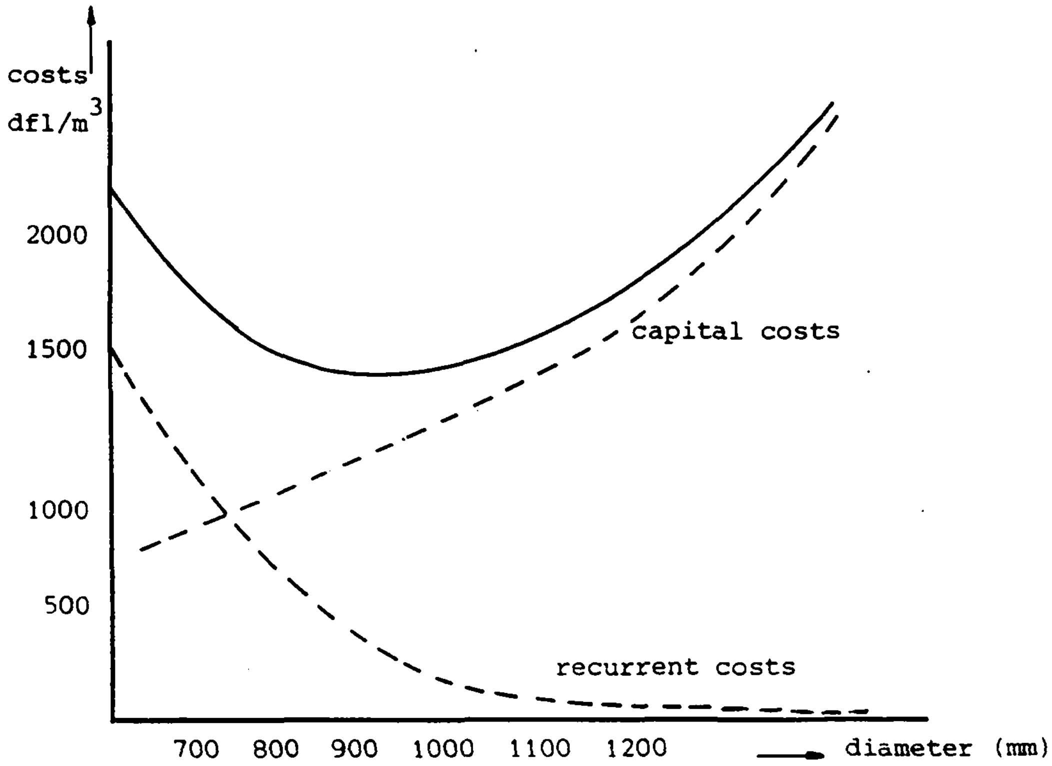


Fig. 17 - Optimization of a transmission line, considering capital costs and recurrent costs

For further information regarding "the most economical design" see appendix 7.

5.2 Example

In order to transport drinking water from a water treatment plant to a distribution area, two technically acceptable solutions are considered (figure 18a.3).

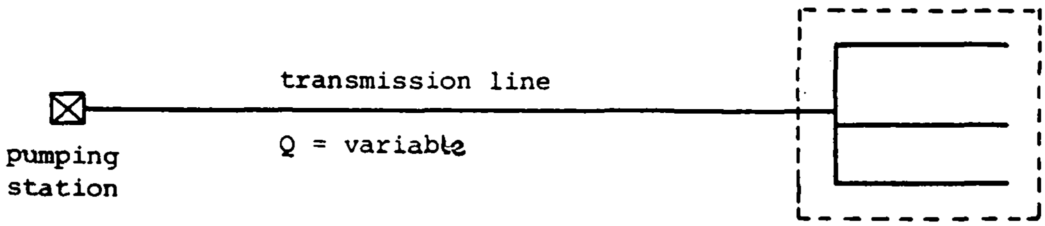


Fig. 18a - Case A: the transmission line is designed for the maximum hourly demand on the maximum day at the end of the design period.

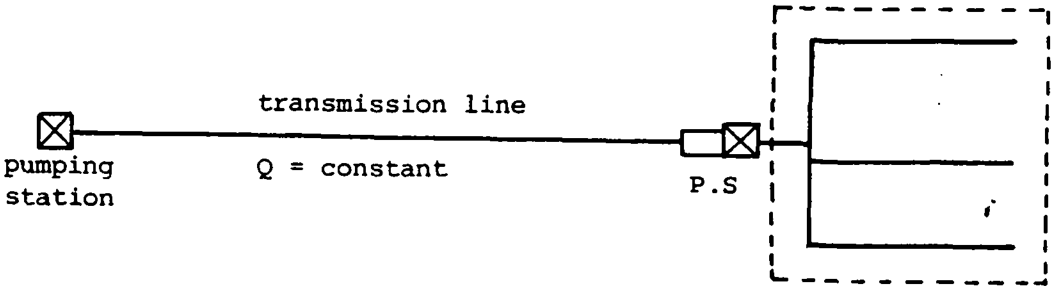


Fig. 18b - Case B: the transmission line is designed for the average hourly demand of the maximum day at the end of the design period, and a storage reservoir is constructed at the end of the transmission line.

Comparing on the basis of costs, it is noted that in the case of alternative B, there is additional investment in a storage reservoir and a pumping station, and a saving in the cost of the transmission line that has a smaller capacity. Obviously, the economic feasibility of case B over case A will depend on the length of the transmission line.

Comparing on the basis of reliability, alternative B has an advantage over A, as in the case of a pipe failure in the transmission line, supply can be continued for some time from the buffer in the storage reservoir. In case of alternative A, supply discontinues abruptly if such failure arises.
Reliability can be increased by constructing the transmission line in two (here equally-sized) pipes. In that case, continuation of supply, although at a lesser maximum flow rate, is guaranteed. Assuming a required diameter of 700 mm in case A, the transmission lines could be constructed as 2 x 600 mm.
In this case the investment in the second main can be delayed to the time when the maximum hourly demand exceeds Q_{A2} (= Q_B).

The pump-characteristic and considered different pipe-characteristics are shown at figure 19.

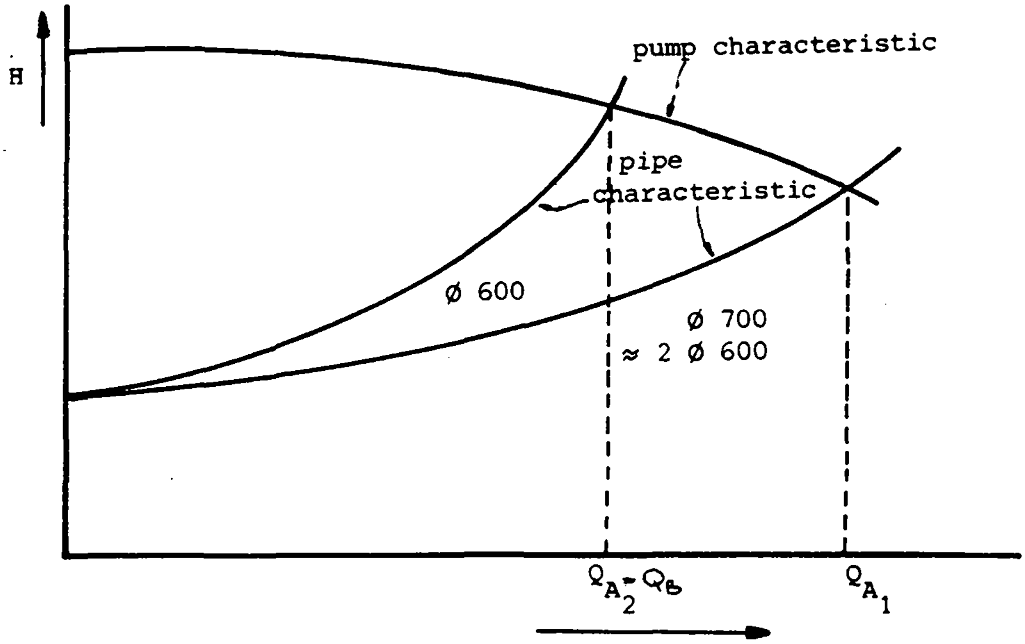


Fig. 19 - Pump-characteristic and considered different pipe characteristics

Q_{A1} = maximum hourly demand at the end of the design period = capacity requirement of the transmission line in case A

Q_B = average hourly demand at the end of the design period = capacity requirement of the transmission line in case B

Q_{A2} = maximum hourly demand at a certain moment during the design period

Note that $Q_{A2} \approx Q_B$.

6 HYDRAULIC FORMULAE FOR NETWORK CALCULATIONS

6.1 Flow formulae

In this chapter some background information on the two most common formulae (Darcy-Weisbach, Hazen-Williams) for network calculations is presented. More information is given in appendix 5.
The formulae define the relationship between flow rates and pressure head losses in a pipeline.

a. Darcy-Weisbach

The formula of Darcy-Weisbach is from the theoretical point of view the most recommendable to calculate the friction losses in a pipesection.

$$\Delta H_{fr} = \lambda \frac{L}{D} \cdot \frac{v^2}{2g}$$

where: ΔH_{fr} = the loss of pressure head in m over a pipesection with length L

λ = the friction coefficient of the considered pipesection

L = the developed length between the two points of the considered pipesection in m

D = the diameter of the considered pipesection in m

$\frac{v^2}{2g}$ = velocity head in m

Substituting $v = \frac{Q}{A} = \frac{Q}{0.25\ \pi D^2}$ and $S = \frac{\Delta H_{fr}}{L}$ the formula may be written as

$$S = 0.0826\ \lambda \frac{Q^2}{D^5}$$

The friction coefficient λ can be calculated by the following formula, derived by Colebrook:

$$(1/\lambda) = - 2 \log ((k/3.7\ D) + (2.51/Re\sqrt{\lambda}))$$

where

D/k = relative wall roughness
D = pipe diameter in m
k = wall roughness in m
Re = Reynolds number = (vD/ν)
v = velocity of flow in m/s
ν = kinematic viscosity in m²/s.

Solution of the Colebrook formula may either be done algebraically or by using the so-called Moody-diagram shown in figure 20.

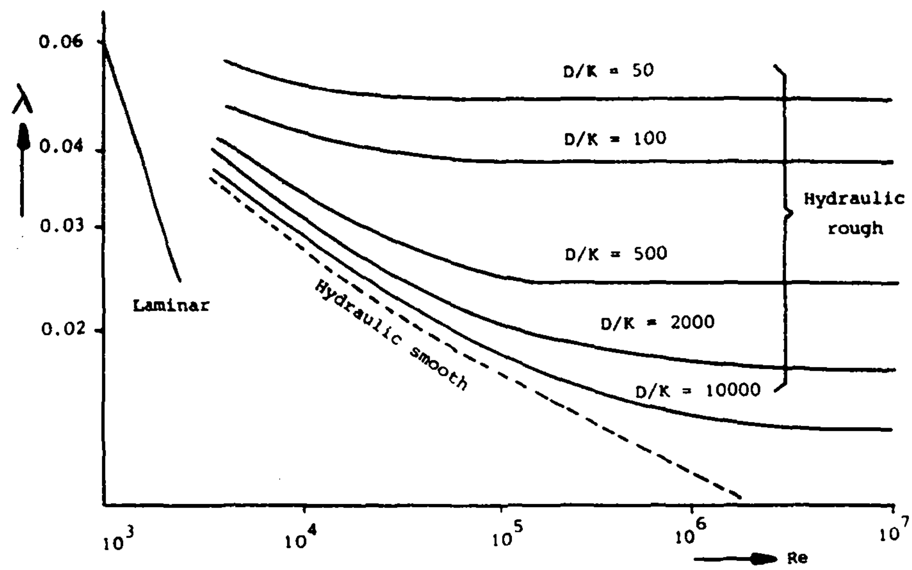


Fig. 20 - The Moody-diagram: determination of the friction coefficient, from the Reynolds number Re and the relative wall-roughness D/k

For pipe velocities between 0.5 and 2 m/s, and pipe diameters between 0.15 and 1.0 m, the Reynolds number is between 10^5 and 10^6 (turbulent flow). As may be seen from the diagram, the coefficient λ for these values of Re is mostly determined by the relative wall roughness D/k and is hardly influenced by the value of Re.

The pressure gradients can also be read from graphs and nomogrammes, where the relationship between pressure gradient, flow rate and diameter is shown for predetermined values of the wall roughness k and the temperature. Figure 21 shows an example of such a nomogramme.

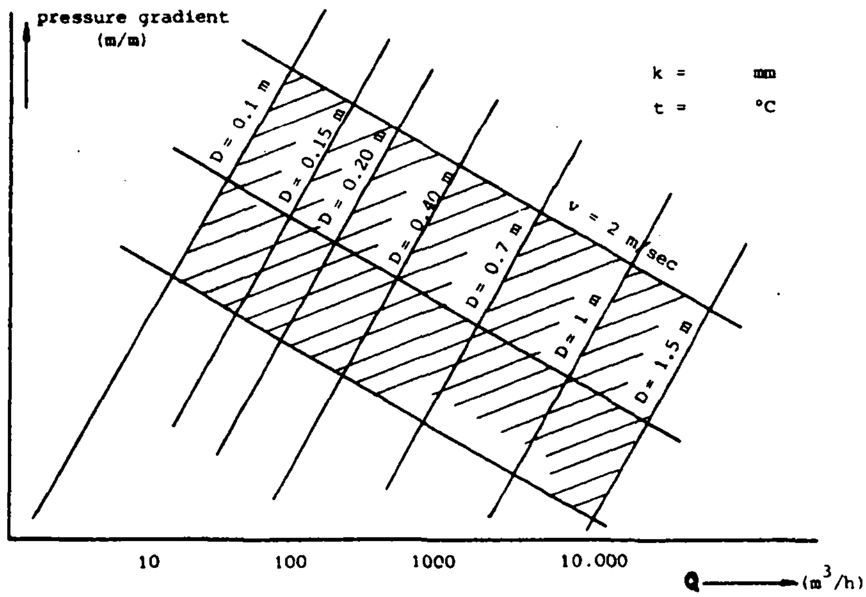


Fig. 21 - The relationship between flow rate, diameter and pressure gradient

The wall-roughness k, normally expressed in mm, is a pipe material constant, and a measure of the hydraulic roughness of the internal pipe

wall. The value for the roughness may be obtained from manufacturers' data sheets or from handbooks.
In steel and cast-iron pipes, the wall-roughness usually increases with age, as corrosion of the steel pipe and formation of incrustations on the cast-iron takes place.
In all pipe materials, precipitation may occur and microbiological slime layers may develop, especially in raw water transmission lines.

b. The Hazen-Williams formula

The Hazen-Williams formula reads:

$$\Delta H_{fr} = 10.26 \times \left(\frac{Q}{C_{HW}}\right)^{1.85} \times \left(\frac{L}{D^{4.87}}\right)$$

or

$$S = \left(\frac{10.26}{C_{HW}{}^{1.85}}\right) \times \left(\frac{Q^{1.85}}{D^{4.87}}\right)$$

where S = pressure gradient
C_{HW} = friction coefficient
Q = flow in m^3/s
D = pipe diameter in m.

In fact this formula compairs quite well with the Darcy-Weisbach formula:

$$S = 0.0826 \cdot \lambda \cdot \frac{Q^2}{D^5}$$

However, where the value for λ depends on wall roughness (k), diameter (D) and flow condition (Re), the value for the friction coefficient C_{HW} is a pipe material constant and is independent of diameters and flow conditions.

Table 7 presents some values for wall roughness k and friction coefficient C_{HW} for use in the Darcy-Weisbach and Hazen-Williams formulae.

pipe materials name	common abbreviation	k-value Colebrook	C-value Hazen-Williams
Polyethylene < 200 mm	HPE	0.01 mm	140
Polyvinylchloride < 200 mm	PVC	0.01 mm	140
Glasfiber reinforced plastic	GRP	0.02 mm	140
Polyvinylchloride ≥ 200 mm	PVC	0.05 mm	130
Asbestos cement	AC	0.1 mm	120
Prestressed concrete (smooth)	PC	0.2 mm	120
Prestressed concrete (rough)	PC	0.5 mm	120
Ductile iron (cement-coated)	DI(C)	0.5 mm	120
Cast iron (cement-coated)	CI(C)	1 mm	100
Cast iron, new	CI	1 mm	100
Cast iron, old	CI	1-5 mm	100-60

Table 7 - Values for wall-roughness k (Darcy-Weisbach) and friction coefficient C_{HW} (Hazen-Williams)

6.2 Examples of hydraulic calculations

a. Turbulent or laminar flow

The condition of flow is turbulent when Reynolds number (Re) is larger than 2100 to 2300.

Using the formula $Re = \frac{vD}{\nu}$ and assuming a kinematic viscosity $\nu = 1.31 \times 10^{-6}$ (T = 10 °C), the velocity above which the flow rate is turbulent can be calculated for various diameters to be:

diameter (mm)	velocity (m/s) above which the flow is turbulent
100	0.03
150	0.02
200	0.015
300	0.01

This means that laminar flows don't occur in networks, where design velocities are between 0.5 and 1.5 m/s.

b. Pressure gradient

The pressure gradient in a 100 mm diameter asbestos cement pipe should be calculated for various discharges using both the Darcy-Weisbach and Hazen-Williams formulae.
First, the velocities and the Reynolds numbers are calculated (T = 10 °C):

discharge m^3/h	velocity m/s	Reynolds number
10	0.35	26700
20	0.71	54200
30	1.06	80900
40	1.41	107600

The value for the relative wall thickness of the pipe, D/k, can be calculated to be 100/0.1 = 1000, and the value C_{HW} = 120 (k and C_{HW} from Table 5).

Using the Moody diagram, the value for λ can be found, and with that the pressure gradient is calculated, as shown below:

discharge	Darcy-Weisbach		Hazen-Williams	
m^3/h	λ	S(m/m)	C_{HW}	S(m/m)
10	0.029	$1.8\ 10^{-3}$	120	$2.0\ 10^{-3}$
20	0.024	$6.1\ 10^{-3}$	120	$7.1\ 10^{-3}$
30	0.023	$13.2\ 10^{-3}$	120	$15.0\ 10^{-3}$
40	0.022	$22.4\ 10^{-3}$	120	$25.6\ 10^{-3}$

7 MATERIALS

7.1 Standardization

A considerable proportion (50 up to 70 %) of the capital investments in water industry is used for the pipes and fittings in the underground system for transport and distribution.
It is obvious that in safequarding a continuous water-supply all pipes, fittings and appurtenances of the transport and distribution system should stand firm.
Within the system the pipe diameter range up to Ø 200 in rural areas and up to Ø 300 in urban areas already covers approximately 80 % of the length of the total network.
The investment cost of the plumbing systems - although not on the account of the water supply company - are enormous too. Totalized it equilizes approximately the investment costs of the distribution system.

A very important factor in water supply systems design, especially in the long range developing programs, is the standardization of materials and equipment.
The first target of standardization always has been an economical one. There used to be rather useless differentation of products caused by independent producing of different manufacturers and the different specifications of clients. So the first step of standardization was elimination of a lot of different types of a certain product. The next step of standardization was a technical one, namely fitness for purpose which has resulted to a quaranteed sufficient quality level for the use of products under

known and evaluated circumstances.
The need for standardization from the point of view of the water supply industry is clear and regarding:

- The planning and design phase
  It allows during designing to be much more precise in specifications.
  The required testing and inspection of materials then gives quarantees for the quality level of the materials which have to be applied.
- The operation and maintenance phase
  A good deal of the difficulties in maintenance come from the diversity of material installed.
  When every new project is bound to bring in still another type of material, problems are inevitable.

The quality level guarantee which is implicit the result of standardization and inspection, is not only important for the water industry as a client it gives at the same time the manufacturer the assurance that the required quality level is fixed and valid for all manufacturers. See also appendix 6 on quality assessment.

Standardization does in general exist for all pipe materials, fittings, appurtenances (valves, hydrants a.s.o.) and all parts of the plumbing systems (check valves, watermeters, taps, a.s.o.).

The next logical step is the generation of standard designs for functional components of a water supply system.
This could start with basics such as public standposts, house and yard connections, low cost plumbing systems.
It could proceed to typical standard designs of

pumping stations, elevated tanks, mini-plants for treatment a.s.o..
Cooperation with the respective local industries in such developments would lead to a much higher degree of self reliance.

Finally such elements could be assembled to ready made projects, allowing for very short preparation periods, accurate cost estimations and uniformity. The feedback of experience in a number of these kind of projects would lead directly to improvements of the standard designs. The advantages in operational management (Operation and Maintenance) and for training and instruction purposes are obvious.

7.2 Pipes, fittings a.s.o.

A typical design may include pipes and fittings as shown in figure 22.

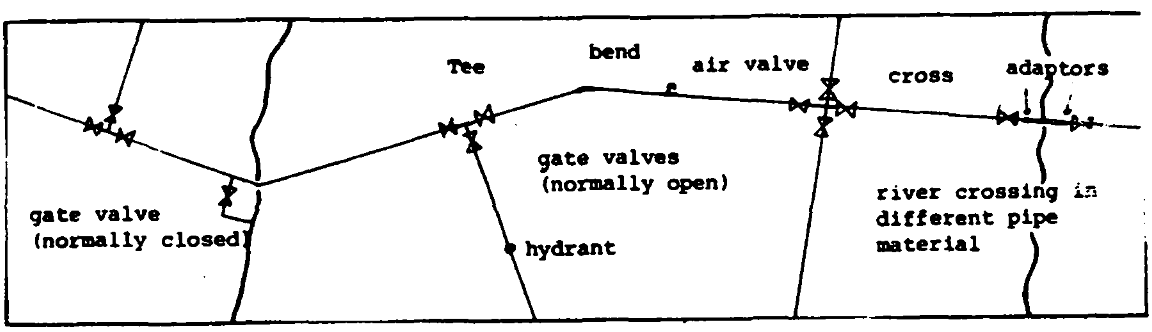


Fig. 22 - Schematic lay-out with different typical pipe fittings

Table 8 gives a general review of the availability of materials for pipes for the transport and distribution of water.

material name	abbre- viation	Ø 100 - Ø 300 mm	Ø 300 - Ø 500 mm	Ø 600 - Ø 700 mm	> Ø 800 mm
asbestos cement	AC	x	x	x	-
polyvinylchloride	PVC	x	x	x/-	-
(high density) polyethylene	HPE	x	x	x	x/-
glassfiber rein- forced plastic	GRP	-	-	x	x
ductile iron (cement-coated)	DI(C)	x	x	-	-
steel	St	x	x	x	x
prestressed concrete	PC	-	-	x	x

Legend x = available
- = not available
x/- = available, but unusual

Table 8 - Availability of pipe diameters

Without presenting detailled specifications of the various pipe materials complete pipeline systems may be distinguished generally in a range from flexible to rigid system. The suitability depends on the actual soil condition. From the soil some lateral support may be expected (clay, sand) with exception of peat soils.

In table 9 a classification of the pipe material and the type of joint is presented.

material abbreviation	common type of joint	pipe material classification	system classification
AC	rubber-ring	rigid	rigid/(flexible)
PVC < 100 mm	glue	flexible	flexible
PVC ≥ 100 mm	rubber-ring	flexible	flexible
HPE	weld, rubber-ring	flexible	flexible
GRP	glue	flexible/rigid	flexible/(rigid)
DI(C)	rubber-ring	rigid	rigid/flexible
Steel	weld	flexible	flexible
PC	rubber-ring	rigid	rigid

Table 9 - Classification of the rigidity of pipe systems

Systems realized in PVC, HPE, GRP and steel pipe material can be considered as flexible, whilst AC and PC are rigid, although the rubber-ring joint some flexibility brings in. Systems in ductile iron may be classified in between.

Typical fittings are joints, tees, bends, reducers, adapters etc.
Table 10 shows which type of materials are used to manufacture fittings for the various pipe materials.

pipes	fittings common	possible
PC	PC	Steel(c)
DIC	DI(C)	--
AC	AC	DI(C); Steel(c)
GRP	Steel(c)	--
PVC	PVC	DI(C); Steel(c)
HPE	HPE	PVC; Steel(c)

(c) = eventually cementcoated

Table 10 - Connection alternatives

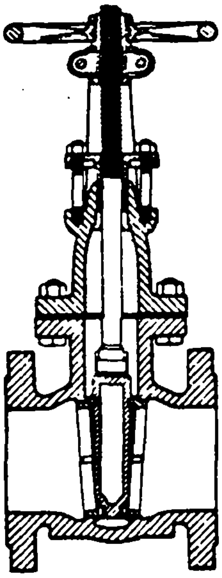
<u>Valves</u>

There are different types of valves, each with its own specific purpose:

- gate valves are used to control the flow rate of water. They are usually fully opened or fully closed (figure 23);
- butterfly valves serve the same purpose as gate valves. They are also used to regulate the flow rate being partly opened (figure 24); -

non-return valves allow the flow of water in one direction only;

- pressure-reducing valves (PRV) and flow controllers. These are used to control pressure or flow rates in a pipeline;
- air valves allow air to escape from the pipeline when filling it and during operation and allow air to enter it when draining the pipe (figure 25).



Gate Valve

Fig. 23 - A gate valve

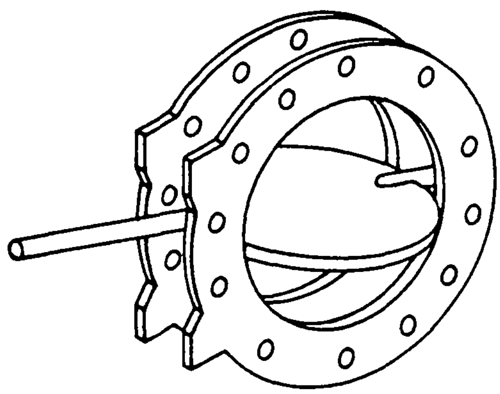


Fig. 24 - A butterfly valve

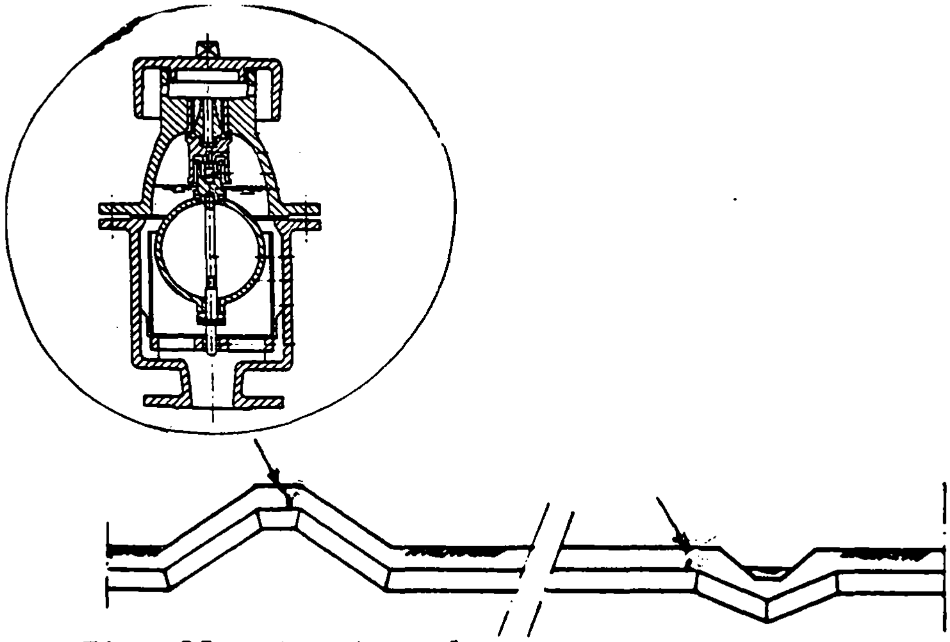


Fig. 25 - An air valve

The choice between gate valves (gate moves perpendicular to flow) and butterfly valves (gate rotates around axis that is perpendicular to flow rate) for shut-off purposes is usually a matter of costs. Butterfly valves are generally cheaper from Ø 300 mm upwards.

Non-return valves prevent backflow by means of a hinged gate, a spring loaded gate or a membrane. In distribution systems a spring loaded gate or a membrane type is used mostly in service pipes and internal plumbing, while the hinged gate type is mounted in the main pipes.

Air valves are mounted on top of the pipe at high points in the pipeline. They are fitted with a floating or springloaded ball that seals off the valves opening to the atmosphere except when air needs to pass.

Fire hydrants

Fire hydrants may be constructed above or below groundlevel. They are installed on main pipes with diameters from 100 mm to 300 mm. Fire hydrants are also used to flush or drain the distribution system (figure 26).

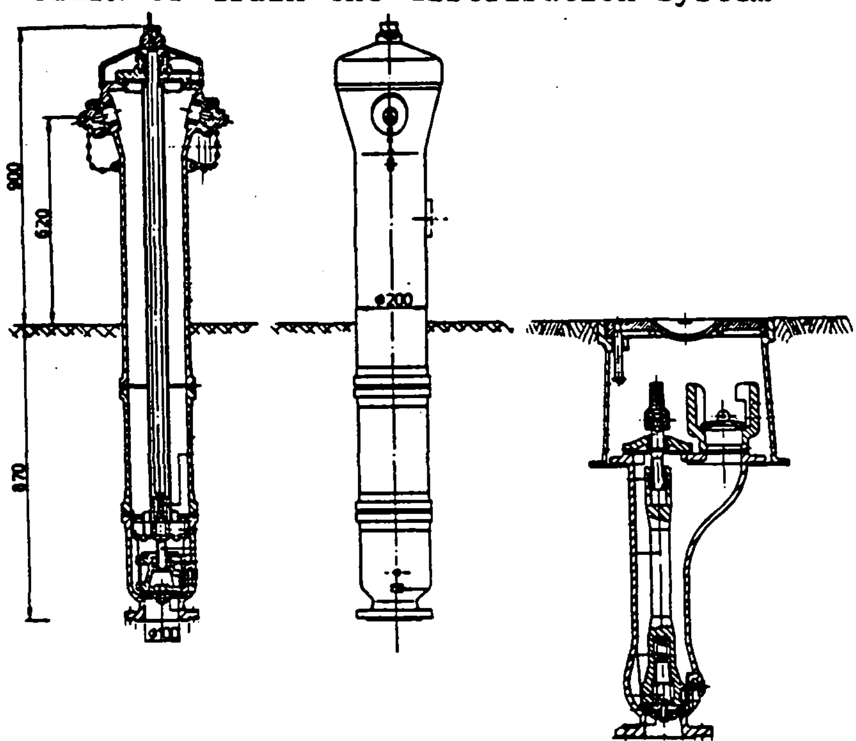


Fig. 26 Overground and underground fire hydrants

Protection of pipes

To prevent the breakdown of a pipeline, due to overloading or corrosion, technical requirements relating to strength and durability of the pipe material must be laid down.

Additionally, the pipe must be given a minimum soil cover to protect it from traffic loads:

- for transmission lines the minimum cover is 1.00 m;
- for main pipes the minimum cover is 0.80 m;
- for service pipes the minimum cover is 0.60 m.

Except for water mains many other conduits are constructed immediately below groundlevel, such as sewers, gas distribution mains (less commonly), electricity and telephone cables. The distance and elevation of water mains in relation to these other conduits should be at least 0.20 m.

With regard to the proximity of sewer lines the groundwater flow should be observed and the water mains should be laid preferably upstream of and at a higher elevation than the sewers.

7.3 Material selection

The selection of suitable pipe materials for a project should be based on technical and financial grounds. The selection should tend to standardization i.e. to a limited number of pipe materials and to a limited number of diameters within the required range. This is of extreme importance as it effects the value and volume of stocks to be maintained, as well as the ease of maintenance of the distribution system. The same applies to fittings etc. to be selected with the pipe material.

In fact, a water supply company preferably should work out an "overall plan", a material policy, based on local conditions (soil condition, costs, local manufacturing, competive supplies, delivery time, service etc.) leading to the selection of pipe materials for use in the entire supply region. It may even be advantageous to consult neighbouring companies to adopt regional or even national policy guidelines with respect to material use.
To ensure technically sound operation on the pipelines, specifications for material purchases should refer to relevant standards for pipes, fittings and appurtenances, so levels are set.

Certain pipe materials may be well combined where other may not. Table 11 shows a number of favourable pipe material combinations, for various diameter ranges. E.g. in the Ø 100 mm ÷ Ø 300 mm range a combination of materials that covers any situation may be PVC + HPE + steel(c) or AC + steel(c) or DI(C) + steel(c).

The remark has to be made that the circumstances, in relation to which the transmission lines and distribution mains are designed, have to be characterized in a general way as favourable or unfavourable. This can be done by according a.o. the soil mechanical properties, the soil agressivity (see appendix 7 for a method assessing soil agressivity) and specific circumstances. The table relates to this for the general part of a network system as well as for specific parts such as street and canal crossings.

Diameter Ø mm	Classification of the circumstances			
	Spec. part	General part		Spec. part
	favourable	favourable	unfavourable	unfavourable
100÷300	HPE	PVC	HPE	Steel(c)
100÷300	DI(C)	DI(C)	DI(C)	Steel(c)
100÷300	Steel	AC	AC	Steel(c)
400÷500	HPE	PVC	HPE	Steel(c)
400÷500	Steel	AC	AC	Steel(c)
600÷700	PC(s)*	AC	AC	Steel(c)
800÷1000	PC(s)*	PC	PC	Steel(c)

* Prestressed concrete with a steel core

Table 11 - Different material selection concepts

Summarized the basic considerations for material selection are:

- the relevant material specifications: appropriate technical requirements, standardization and a selected material range;
- the required technical application;
- the cost comparison;
- the policy check.

On base of appropriate technical requirements a guaranteed quality level of the mentioned preselected materials is possible.
A further selection can be made on base of the local circumstances and required specific technical alternatives.
The final choice however is done after a check on the material policy f.i. preference for local production. This preference doesn't exclude the first mentioned minimal technical requirements and thus the required minimal acceptable quality level.

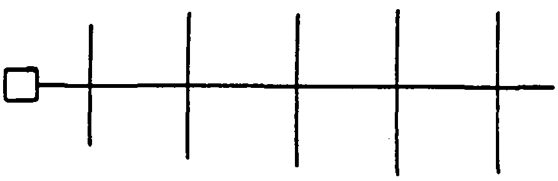
8 DESIGN PROCEDURES

8.1 Distribution network configurations

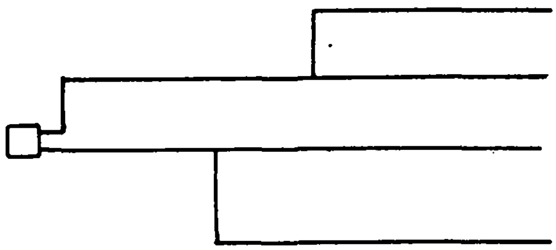
In order to ensure a sufficient quantity of drinking water and a sufficient pressure at each point of a distribution area, a well-designed distribution network is required. There are two basic network configurations:
- the branch system,
- the grid system.

There is also a variety of combinations.

A branch system can be designed with trees, parallels, or a combination of both. (See figure 27.)



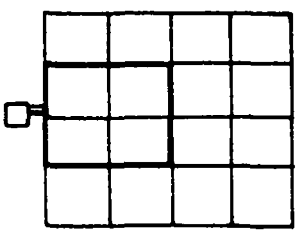
a tree-type



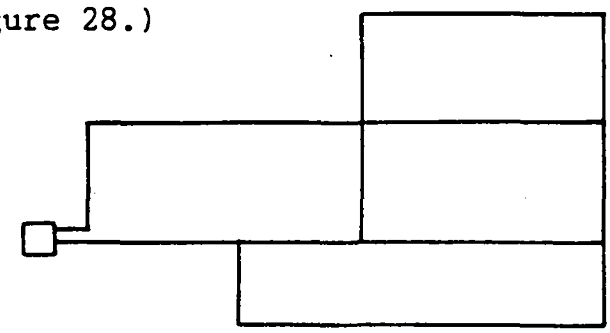
b parallel-type

Fig. 27 - Branch systems

A grid system can be designed as a looped system or an extension of the branch system by means of extra connections. (See figure 28.)



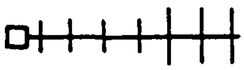
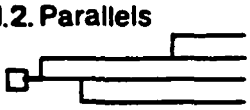
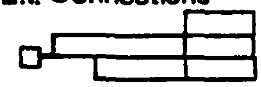
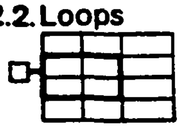
a looped-type



b connected parallel-type

Fig. 28 - Grid systems

In table 13 the advantages and disadvantages of the above-mentioned systems are presented in relation to their appropriateness in rural and urban areas, the reliability in general, the probable influence of the configuration on the water quality and the investment costs.

	Relia-bility	Water-quality	Invest-ments	Appropriateness Urban	Rural
1. BRANCH system					
1.1. Trees 	−	+/−	+	−	+
1.2. Parallels 	−	+/−	+/−	−	+
2. GRID system					
2.1. Connections 	+	+	+/−	+	+/−
2.2. Loops 	++	+/−	−	+/−	−−

Legend: + = convenient
- = inconvenient

Table 13 - Review of characteristics regarding possible network configurations.

Branch system

Branching is in principle an adequate method of distribution.

The design of a branched network is simple; there is no change direction of the flow rate and the required investments are relatively low compared with the grid system.

However there are some disadvantages such as:

- sediments may accumulate due to stagnation of the drinking water at the dead ends, occasionally causing taste and odour if the pipes are not regularly flushed;
- the reliability of supply is low (and there is a danger of contamination); during repairs large parts of the network may be shut off and therefore without water;
- future extensions may cause pressure problems.
- a fluctuating water demand results directly in rather high pressure fluctuations.

Grid system

The grid system, based on the branch network with connected parallels, decreases the above-mentioned disadvantages of branch systems:

- the water in the system flows in more than one direction and lasting stagnation does not occur as easily as in the branch system;
- in case of pipe bursts and during subsequent repairs, the area concerned will continue to be supplied by water flowing towards it from the opposite direction. A pressure increase at the pumping station improves this;
- there will be a smaller adverse effect on the supply of water due to large fluctuations in water demand.

A disadvantage of the grid system is that the required investment costs are (somewhat) higher than in the case of a branch system.

The looped type grid system is appropriate especially and exclusively for very important urban and industrial areas. Loops can be provided within a grid pattern to improve waterpressure in sections under all circumstances.
Loops should "moreover" be stratigically located so that ongoing town development the water pressure can be maintained.
The advantages of the looped-type system are as mentioned for the connected parallel-type of grid systems; the realiability is very high, but the investment costs are considerably higher too.

8.2 Schematization

Larger urban distribution systems are usually grid-type systems. They may have been developed from branch systems, however. When designing the initial phase of a growing network, one must consider first starting with the (cheaper) branch system, which may, during later extensions, be converted into a grid system. An alternative design procedure is to start with the final grid system directly but reduce this to an adequate branch system for the first stage of operation.

In rural areas, regional and local distribution systems are usually of the branched type. In the regional systems the demand of each settlement is represented in one or two nodes of the schematized network.

Calculations of networks are relatively simple for branch systems and more complicated for grid systems due to the number of computations involved.

Generally speaking, the network is schematized prior to calculations, e.g. by:
- concentration of the demand at the nodes (a node is the conjunction of two or more pipes);
- neglecting the relatively small pipes, because of their relatively small capacities (and for that reason the very small influence on the results of the network calculations);
- the introduction of equivalent pipe diameters and/or (equivalent) lengths for computation purposes.

8.3 Design methods

There are two main methods for the manual calculation of distribution systems, the equivalence method for the calculation of branched systems (see Chapter 9) and the Hardy-Cross method for the grid systems (see Chapter 10).

The manual application of the Hardy-Cross method is limited to smaller systems (say: less than four loops). With larger systems the calculation becomes very time-consuming and less accurate. Larger grid systems are therefore calculated with the use of a computer with specially designed software for network calculations (see WATER TRANSPORT AND DISTRIBUTION - Part "Computer Applications for Distribution".

In fact, the computer is being increasingly used in network design. Apart from the calculation of larger grid systems, the computer may be used to check earlier manual calculations and to make additional calculations. These additional calculations are made to check the operation and reliability of the system under special circumstances,

i.e. when fire-fighting and during pipe line breakdowns. Also various extreme demand situations may be simulated. In this way, the computer is used to finalize the design.
In the finalization of the design, the effects of the inclusion or exclusion of certain connections and the enlargement or reduction of the diameters of certain pipesections are studied by calculating resulting pressure gradients and pipe velocities and minimum and maximum pressures within the network for various alternatives.

The design procedure therefore progresses as follows:

1. manual computation using the equivalence method for a branch system and the Hardy-Cross method for a grid system;
2. optimization of the design by well-chosen improvements, followed by additional manual calculations;
3. checking of the system on reliability by simulating certain calamities. These calamities shall not result in unacceptable situations. Additional investments must be weighted against increased reliability. This phase may be executed using a computer.

8.4 Example

A very simple design example of a branch system is presented below.

The situation (figure 29) is as follows:
A pipe AD is connected to a transmission line with guaranteed pressure of at least 40 m.w.c. The

ground levels at A and B are zero and at C and D respectively 8 m and 1 m above sea level.
Along pipe AD dwellings are located. The total number of dwellings is 560 with an average occupancy of 5 capita per dwelling. This means a total number of inhabitants of 2800.
The average demand is 100 l p.c.p.d. with a peak factor of 2.4. Simultaneity is neglected.

In addition to the domestic demand a small fire fighting flow of 20 m^3/h may be required, at any time.
The minimum pressure requirement is 10 m.w.c. above ground level.

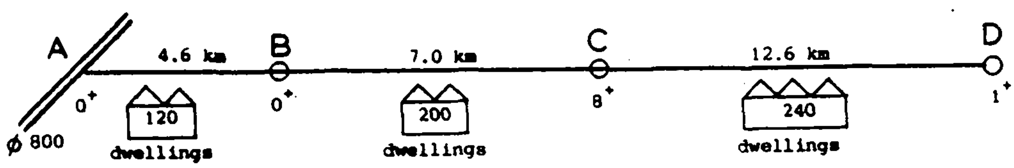


Fig. 29 - Situation for sample calculation of a distribution main.

First, the situation is schematized. The 120 dwellings along section AB are relocated as follows: 60 at A and 60 at B. Likewise, the dwellings along the other sections are relocated at nodes B, C and D. This results in 60 dwellings at A, 160 at B, 220 at C and 120 at D.

The peak demand for an average dwelling is 2.4 x 0.1/24 x 5 = 0.05 m^3/h. The schematized peak domestic demand at the 4 nodes is: 3 m^3/h at A, 8 m^3/h at B, 11 m^3/h at C and 6 m^3/h at D. The most unfavourable location for the 20 m^3/h fire-fighting requirement is D, as in that case the flow rate has to pass the entire section AD.

The resulting design flow rate in the various pipe sections can be calculated from the water demand at the various nodes, starting at D. In D the demand is 26 m^3/h. The design flow rate for section CD is therefore 26 m^3/h. In C, the demand is 11 m^3/h. Section BC must pass the demand at node C, plus the design flow rate in section CD, totalling 37 m^3/h. Likewise, the design flow rate for section AB is 45 m^3/h and the total demand for branch AD, to be delivered at A, is 48 m^3/h (see also figure 30).

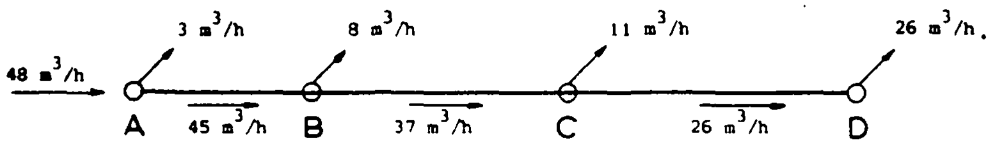


Fig. 30 - Calculated design flow rates in the various distribution main sections

The maximum pressure loss over the distance AD is (40 - 10 - 1) m.w.c. = 29 m.w.c., where
40 m.w.c. is the minimum pressure at A
10 m.w.c. is the required minimum pressure at D and
1 m.w.c. is the difference in elevation between A and D.

As the distance AD is 24.2 km, the maximum permissible pressure gradient is 29/24.2 = 1.2 m/km.

Using graphs such as those shown in appendix 5, pipe diameters can be selected for which the pressure gradient corresponds to the above value for the calculated design flow rate. The use of the tables requires prior determination of the pipe material (k-value) and the water temperature (kinematic viscosity). The following is a possible solution:

Pipe section	Length [km]	Diameter [mm]	Discharge [m^3/h]	Pressure gradient [m/m]	Pressure loss [m]
CD	12.6	150	26	0.0015	18.9
BC	7	200	37	0.0007	4.9
AB	4.6	200	45	0.0010	4.6

Pressures at ground level may now be calculated starting from A as follows,

at B 40 - (4.6) = 35.4 m

C 40 - (4.6 + 4.9) - 8 = 22.5 m

D 40 - (4.6 + 4.9 + 18.9) - 1 = 10.6 m

The water pressure at D equals the pressure at A (40 m.w.c.) less the total pressure loss over sections AB to CD (28.4 m), and less the difference in elevation between D and A (1 m) (see also figure 31).

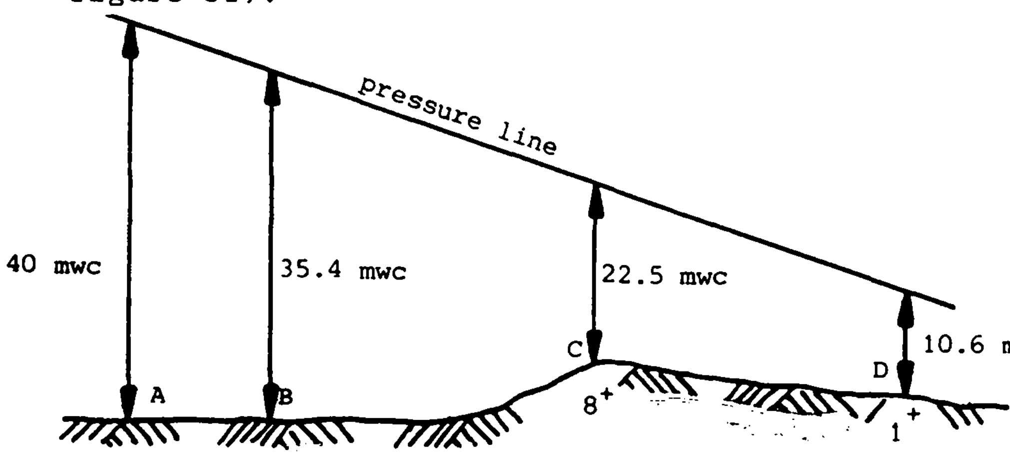


Fig. 31 - Pressure line and pressure heads along the distribution main

9 EQUIVALENCE METHOD

9.1 Procedure

The equivalence method relates the number of inhabitants (or hectares) supplied by a certain pipe directly to a design diameter for that pipe. The so-called equivalence tables used for this purpose should be specially designed (or selected) for each situation. See for the use of the method in practice also appendix 8.

The equivalence method can be used for the calculation of branched type networks only. A branched system in general is more economic than any other system.
Grid type systems can also be calculated with this method, however only after converting the grid system in a branched type system by cutting the pipes expected to carry a low discharge (see figure 32).

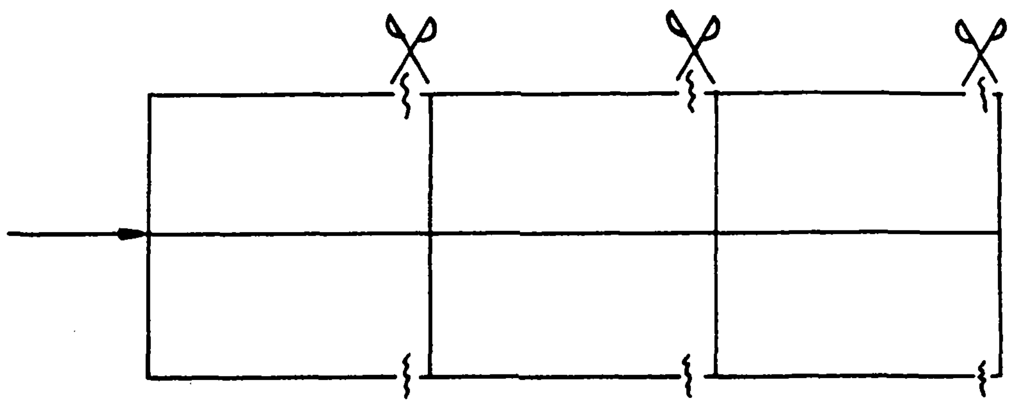


Fig. 32 - The conversion of a grid type network into a branched system

In cutting up a grid type system, the following rules of thumb apply:

1. assume the demand to be concentrated at the nodes;
2. the route to be followed by a water particle should be the shortest possible one;
3. all pipes should be dimensioned such that they operate at full capacity, i.e. at maximum permissible gradient (or velocity).
4. the unit cost of water transport decreases with increasing pipe diameter.

9.2 Equivalence table

In an equivalence table there is a direct relation between inhabitants, or hectares, or population equivalents (p.e.) on the one hand, and pipe diameters on the other hand.

The equivalence table used in the branched system design should be based on careful consideration of the situation in which the distribution system is to be constructed. In the equivalence table shown in Table 14, inhabitants are related to pipe diameters through the capita demand, peak factor, simultaneity factor, permissible pressure gradient (or velocity), water temperature and pipe material. As these factors may differ for various situations, so do the resulting equivalence tables.

Table 14 is based on the following values of the above-mentioned factors:

per capita water demand:	q = 150 l/p.c.p.d.
peak factor	: p.f. = 2.25
permissible gradient	: s = 1 to 4 m/km
water temperature	: T = 10 °C
pipe material	: DI(C) (k = 0.5 m)

Ø mm	Permissable pressure gradient m/km	Capacity m³/h	Relative capacity -	f_{mom} -	Q_{max} per capita l/h/cap	Capacity in number inhabitants inh.
100	4	16	1	5	30	500
150	2	32	2	3	20	1.600
200	2	70	4.4	2.3	15	4.700
300	1	140	9	2.25	14	10.700
400	1	300	19	2.25	14	21.500
500	1	540	34	2.25	14	38.500
600	1	850	53	2.25	14	61.000

Table 14 - Example of an equivalence table

9.3 Example

A new residential area is to be supplied with potable water. Figure 33 shows the location of blocks of apartments with 250 (type a), respectively 400 inhabitants (type b).

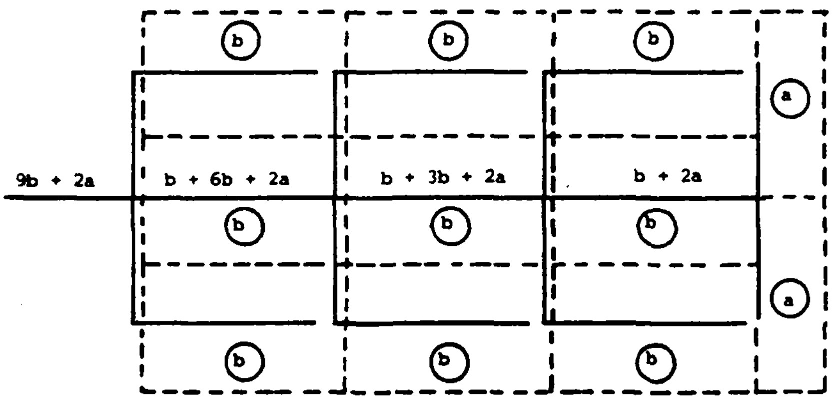


Fig. 33 - Lay-out of appartment blocks with a possible branched distribution system

Following the same schematization procedure as in the earlier example, the design flow rates expressed in inhabitants are as shown in figure 34.

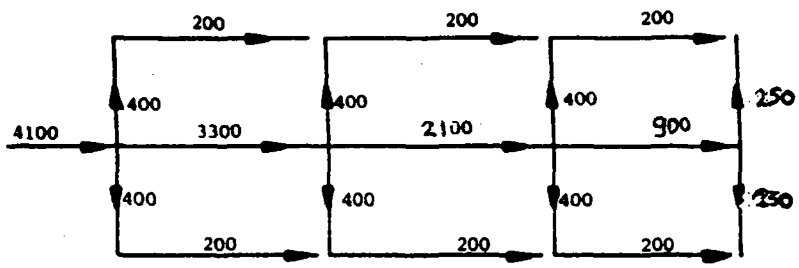


Fig. 34 - Design flow rates expressed in inhabitant-equivalents

Assuming the equivalence table of Table 12 to be applicable, the pipe diameters would be as shown in figure 35.

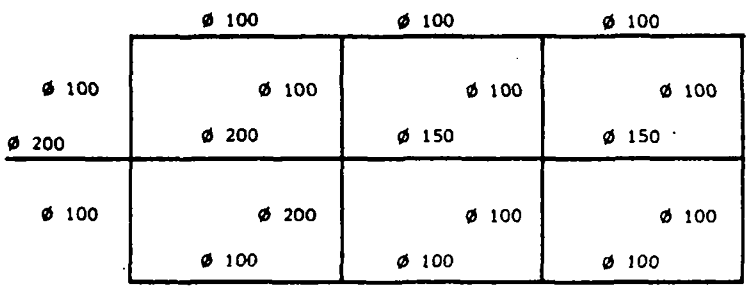


Fig. 35 - Pipe diameters in mm resulting from equivalence table 14

Figure 36 shows an alternative lay-out for the branched distribution system, with design flows.

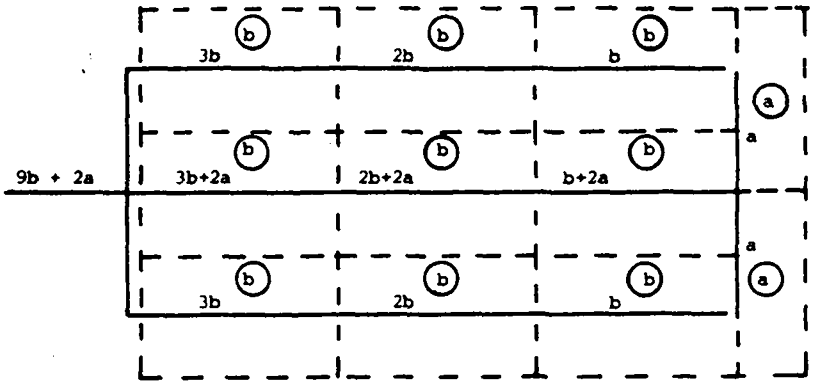


Fig. 36 - Alternative lay-out of the distribution system

Using table 14, the diameters for the pipe are now as shown in figure 37.

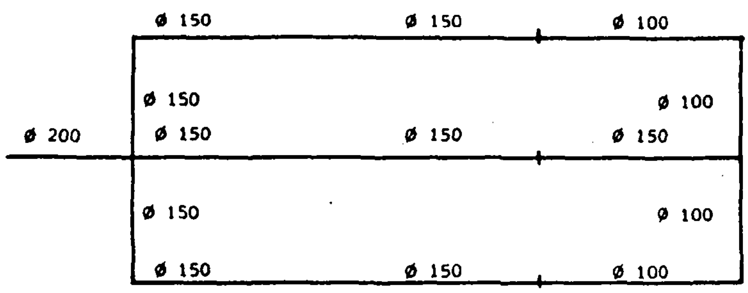


Fig. 37 - Pipe diameters in mm for the alternative solution

Obviously, the reliability of the alternative solution is less and may be increased by one cross-section. The selection between the solutions may be based on economical grounds, provided both solutions are technically acceptable.

10 HARDY-CROSS METHOD

10.1 Background

The Hardy-Cross iterative method is often used to determine heads and flows in a pipe network. Cross, in fact, suggested two different iterative procedures.
The first procedure is called "The Method of Balancing Flows"; the other, "The method of Balancing Heads". Both methods are based on the hydraulic laws of Kirchhoff.

1st Law of Kirchhoff (Continuity equation in an node): The algebraic sum of all ingoing and outgoing flows in a node is zero. Σ Q = 0 (figure 38).

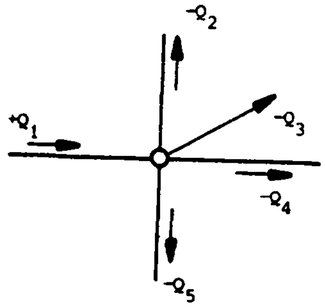


Fig. 38 - 1st Law of Kirchhoff applied to a node

2nd Law of Kirchhoff: The algebraic sum of all head losses in a grid should be zero. ΣH = 0 (figure 39)

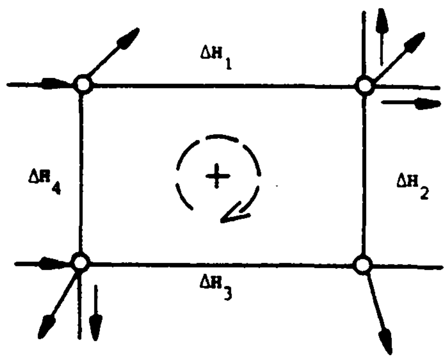


Fig. 39 - 2nd Law of Kirchhoff applied to a loop

10.2 The method of balancing flows

Theory and application of this method are shown in detail in appendix 5.

The procedure is as follows: first, the diameters of the pipes in the grid system are estimated. Next, the flow rates in the pipes are estimated. From these two data, and knowing the pipe length, material and water temperature, the pressure loss in each pipe of a loop can be calculated. The sum of these losses should be zero in the loop (2nd Law of Kirchhoff). If it is not, and it usually is not, a correction flow is sent through the loop in an effort to get the sum of head losses near zero. The value of this corrective flow equals:

$$\Delta Q = - \Sigma\Delta H/2\ \Sigma\left|\frac{\Delta H}{Q_i}\right|$$

After several corrections, the sum of the head losses is near enough zero and the actual pipe flows are the estimated flows corrected with the $\Sigma\Delta Q$ from the successive iterations.

10.3 The method of balancing heads

Theory and application of this method are also shown in detail in appendix 9.

The procedure is as in the case of balancing flows: but now, pressure heads are first estimated. With selected diameters and other data, flow rates to and from a node can be calculated. These should add up to zero. The required corrective head equals:

$$\Delta H_i = 2 \sum_{j=1}^{n} Q_{ij} / \sum_{j=1}^{n} \frac{C_{ij}}{\sqrt{\Delta H}}$$

where $C_{ij} = \frac{1}{\sqrt{f}}$, from $\Delta H = fQ^2$ or $Q = \frac{1}{\sqrt{f}} \sqrt{\Delta H}$

After several corrections, when ΣQ_i is near enough zero, actual pressure heads can be calculated from the original estimates corrected for the various ΔH's used in the iterations.

10.4 Procedure

The various iterations are executed with the help of a table where the results of the calculations are noted. The tables are shown in appendix 9.

The computational procedure for the method of balancing flows proceeds as follows:

1. preparation of an appropriate table for the calculation of each loop;
2. completion of the table with the available data, such as loop number, pipe section name, pipe length and diameter;
3. assumption of pipe flow rates;
4. calculation of pressure losses, and the sum of the losses in each loop. Calculation of corrective ΔQ;
5. calculation of new pipe flow rates;
6. repetition of (4) and (5) with the new pipe flow rates until the sum of the losses is near enough zero.

The procedure for the method of balancing heads is of course almost simular.

11 RELIABILITY OF NETWORKS

11.1 Technical measures

The reliability of a networks may be defined as the probability that it will, throughout the design period perform in accordance with the criteria as laid down in the design.
Ideally, the reliability should be 100 %, but in practice it is less.
Some measures that are known to increase reliability are standard practice (however, not in all countries). Others are subject to cost-benefit analysis first.

Examples of standard measures are:
- any distribution area is supplied from more than one side;
- the inclusion of technical provisions such as twin transport mains, connections, valves, hydrants, etc.;
- the use of appropriate construction materials;
- good quality workmanship in design, construction and operation and maintenance.

The inclusion of connections, valves and hydrants will be discussed in some more detail.

- Connections

  By providing twin transmission lines with connections, the transport capacity may be largely maintained in case of a burst in one section, whereas a pipe burst in a non-sectioned main would, assuming fixed energy input, reduce the transport capacity of a single line to zero and of a double line to half of the original capacity.

The sectioned twin line could maintain a capacity of 75 % or more, depending on the design (see also figure 40).

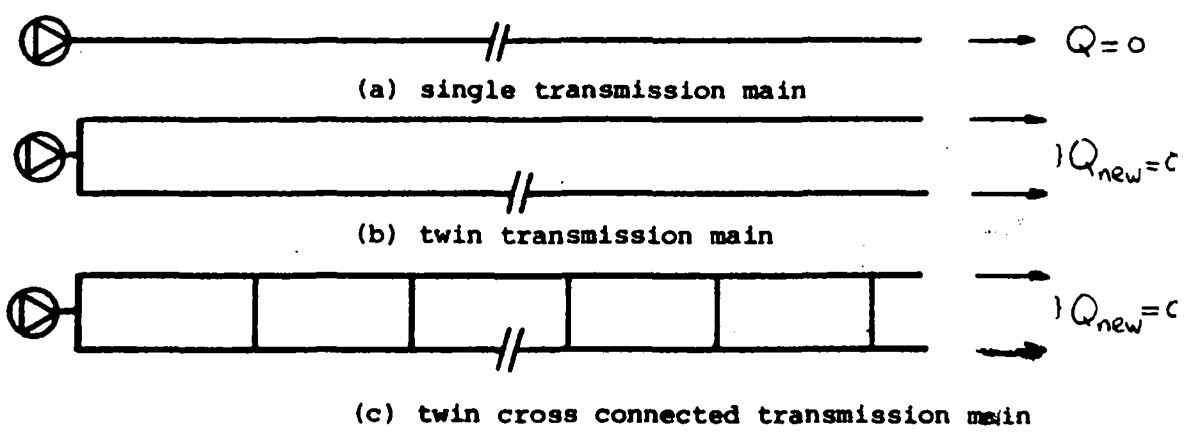


Fig. 40 - Reduction of transport capacity due to a pipe burst depends on the design

In long transmission lines, connections are constructed every 4 to 5 km; in distribution mains, connections are made every 300 to 500 m in urban areas and every 1000 m in rural areas. (See figure 41).

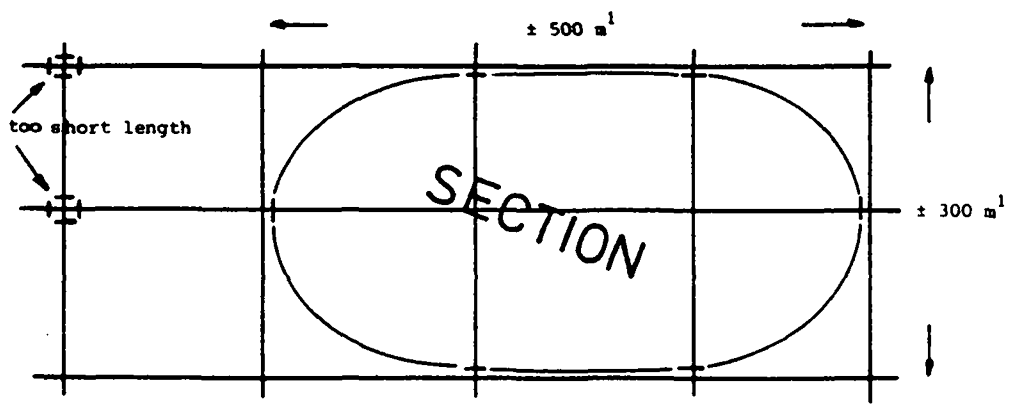
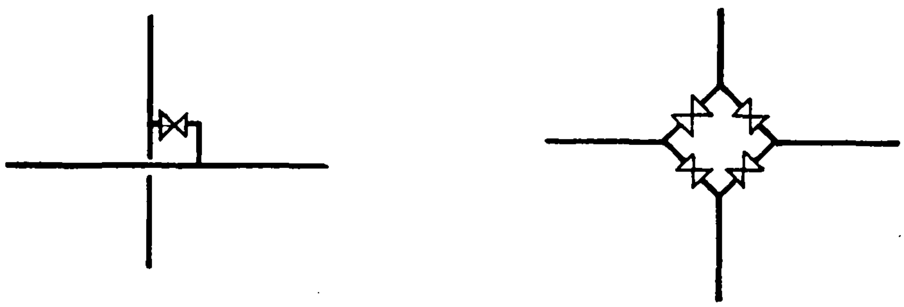


Fig. 41 - Connections in a grid type distribution system

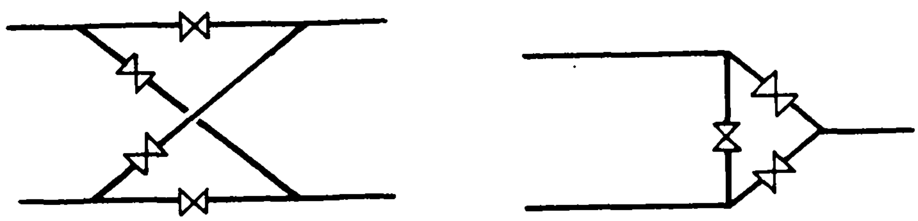
- Gate valves

Gate valves are provided in transport and distribution systems to make it possible to prevent or allow the flow of water in certain sections. The connections shown in the above figures 25 and 26 are provided with gate valves that are closed to prevent continued outflow of water in case of a burst in the section.

Several types of connections and the location of the valves are shown in figure 42.



a. Crossing b. Complex crossing



c. Connection of two parallel mains d. Junction

Fig. 42 - Some specific pipe connections with the location of gate valves

- Hydrants

The principal goal of a distribution system is to supply drinking water to the urban or rural population. Another goal may be to supply fire-

fighting water.
The provision of fire-fighting water in a community may be by open channels, ponds, or special fire-fighting reservoirs, or by fire-hydrants that are part of the distribution system. In the latter case the design criteria for fire-fighting in terms of flow rate and pressure must be formulated, and the distribution system should be checked for its ability to satisfy these criteria.

In areas with high water demand (such as high density residential areas, commercial or industrial areas) the network in general has sufficient capacity to cope with fire-fighting requirements. In areas with smaller demands, however, pipe diameters may need to be enlarged to cope with fire-fighting design flow rates. In that case, the provision of fire-fighting facilities is more costly, as, with the exception of the hydrants, there are more additional costs for pipes and fittings.

Once installed, fire hydrants can also serve as laboratory sampling points, to flush the distribution system, and to distribute drinking water in case of emergency.

Hydrants are usually placed 200 to 300 m apart, suitably located near street corners and in strategic locations, in consultation with the fire service.
Design criteria for fire hydrants may differ, but generally a flow rate of 60 m^3/h at a pressure of 10 m.w.c. above street level is required. The minimum size of the distribution main in which

the hydrant is placed is generally 100 mm, whilst the main should preferably be supplied from two sides. Hydrants are not normally placed on mains with a diameter above 300 mm.

11.2 Failure to supply high demand

A distribution system is generally designed to provide the maximum hourly demand. With respect to reliability of supply, it is interesting to see how often this maximum hourly demand occurs. In figure 43 an example of the frequency distribution of the hourly demand over the year is shown.

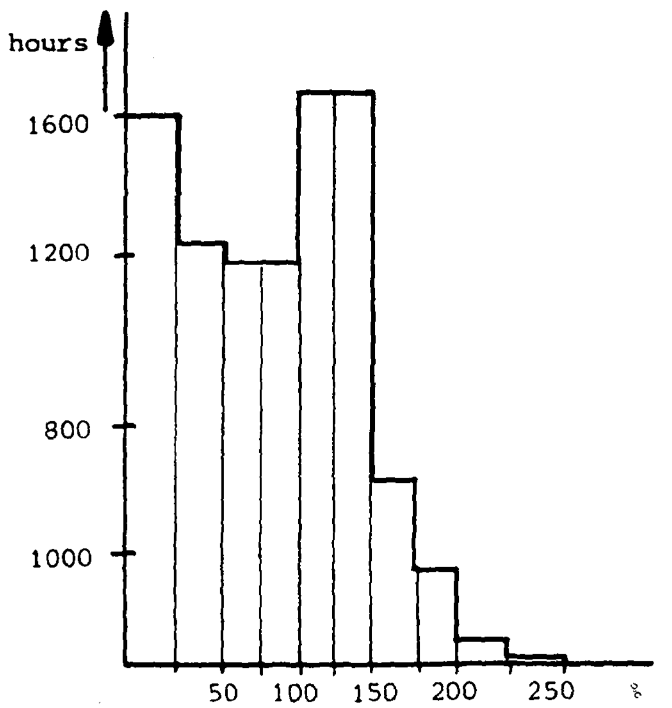


Fig. 43 - Frequency distribution of hourly water demand over the year

From this curve a cumulative frequency distribution curve can be derived showing the number of hours in the year that the demand is in excess of a certain percentage of the recorded maximum hourly demand (see figure 44).

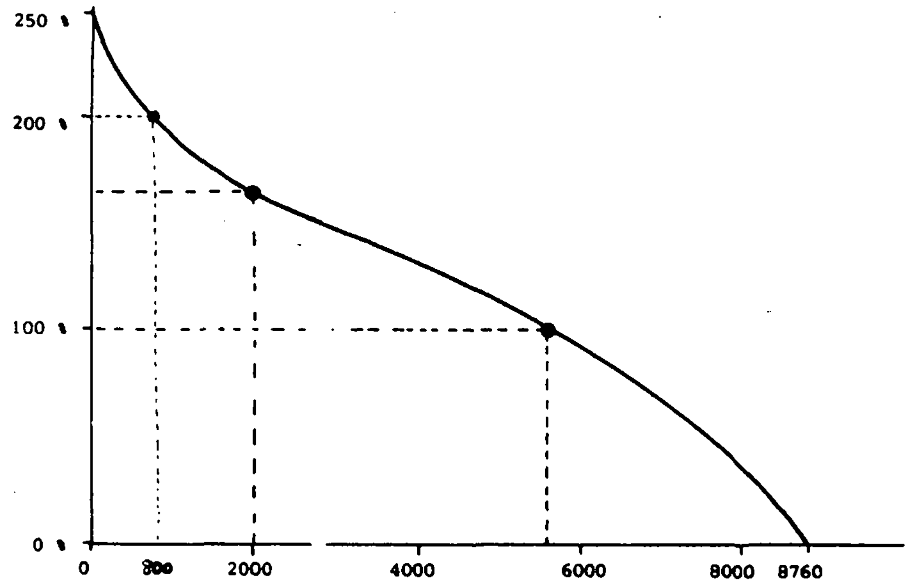


Fig. 44 - Cumulative frequency distribution of hourly water demand in one year

In the example used to draw up figures 43 and 44, as may be seen from the graph, the demand rises above 80 % of the design demand (maximum hourly demand) for fewer than 800 hours (10 % of the annual number of hours) only.

11.3 <u>The occurrence of pipe bursts</u>

At the 9th International Water Supply Association Congress in New York, a French paper presented a graph relating the number of pipe bursts to the diameter of the burst pipe. This graph shown in figure 45 was checked and found valid also for the situation in The Netherlands. General (global) validity cannot, however, be accredited.

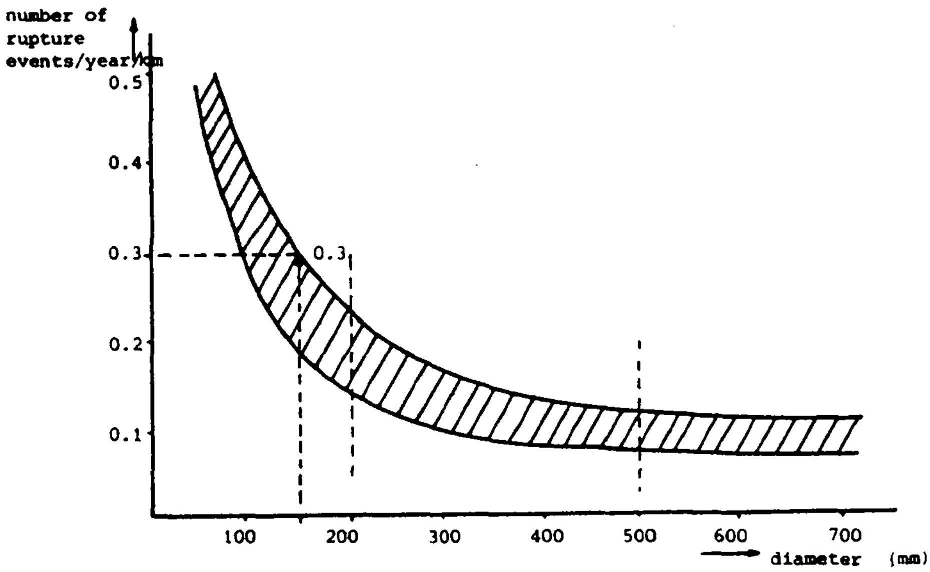


Fig. 45 - The number of burst events per year per km related to the pipe diameters

Examples

The probability of a burst in a Ø 400 transmission line of 1000 m' and a connected Ø 150 distribution main of 400 m' can be read from the graph (figure 45).
The burst event probability of the distribution mains is 0.4 x 0.3 = 0.12.
Together these 1400 m' of network has a probability of a rupture of 0.27, i.e. once every 4 years. The repair time of the Ø 150 and Ø 400 mains will determine whether the interrumption is acceptable or not in the given situation.

Another example is given with help of the section or district of figure 41 paragraph 11.1).
All distribution mains are assumed to be Ø 100 and the total length within the total district is 1100 m'.

The probability of a rupture causing the total separation of the district from the network is 0.3 x 1.1 = 0.33, i.e. once every three years. This seems to be very acceptable. In emergency cases the distance to the network which is not affected is about 200 m'.

APPENDICES

LIST OF SYMBOLS AND UNITS

Description	Abbre-vations	S.I. Unit
Length	L	kilometre [km] metre [m] millimetre [mm]
Area, cross section area	A	square metre [m²] hectare (10.000 m²) [ha]
Diameter	D	metre [m] millimetre [mm]
Mass	M	kilogram [kg]
Volume	V	cubic metre [m³] litre (0,001 m³) [l]
Time	t	second [s] day [d] year or annum [a]
Discharge, flow rate	q or Q	cubic metres per second [m³/s] litres per second [l/s]
Velocity	v	metre per second [m/s]
Force	F	Newton [N]
Pressure, pressure head	P	Pascal [Pa] (1 pascal = 1 N/m²) metre water column (1 m.w.c. ≈ 10 kPa)
Kinematic viscosity	ν	stokes [St] square metres per second [m²/s]
Temperature	T	degree Celcius [°C] degree Kelvin [°K]
Energy	E	Joule [J] (1 Joule = $2.778 * 10^{-7}$ kWh)
Power	N	watt [W] kilowatt (1000 W) [kW] (1 watt = 1 joule per second)

LEGEND FOR DRAWINGS

Pump :

Trunk main :

Distribution main:

Service pipe :

Gate or butterfly valve:

Non return valve :

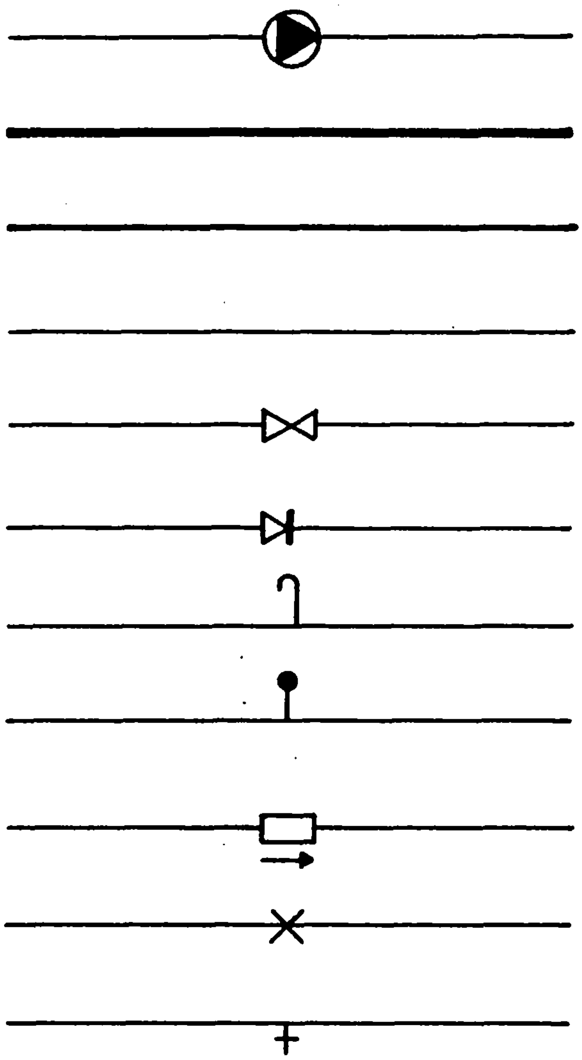
Air valve :

Fire hydrant :

Water meter :

Stopcock :

Draw off tap :



PRESENT VALUE METHOD

A good method for comparing alternatives is the Present Value Method or Present Worth Method. Within this method all actual investments and future investments as well as annual costs are calculated (back) to a reference year (which will be in general the year of the first investments).

Some definitions and formulae

Single Compound Amount Factor (s_{nr}):
The s_{nr} gives the growth of a present sum P in n years with an interest rate of r % and with a compounded interest.

$$s_{nr} = (1 + r)^n$$

Single Present Worth Factor (p_{nr})
The p_{nr} is needed to calculate the present worth of a future sum to be available at year n and with an interest rate of r %.

$$p_{nr} = \frac{1}{s_{nr}}$$

The present worth PW of a future sum F_{nr} is obviously

$$PW = \frac{F_{nr}}{s_{nr}} = F_{nr} * p_{nr}$$

Capital Recovery Factor or Annuity Factor (a_{nr})
The a_{nr} times a present P is equivalent to an uniform series of n annual sums A at an interest rate of r %.

$$a_{nr} = \frac{r * (1 + r)^n}{(1 + r)^n - 1}$$

$$A = P * a_{nr}$$

Uniform Present Worth Factor (UPWF)

The UPWF is needed to calculate the present worth of an uniform series payments A over n years and at an interest rate of r %.

$$UPWF = \frac{1}{a_{nr}}$$

The present worth PW of a uniform series of annual payments A over n years and at an interest rate of r %.

$$PW = A * \frac{1}{anr}$$

Remarks:

- If instead of the interest rate per annum (= true interest rate) other interest periods are used these should be corrected.

- Inflation is not taken into account yet.
  The introduction of a so called ideal interest rate (i) gives this opportunity

  $$i = \frac{1 + r}{1 + f}$$

  If for instance the true interest r is 8 % and there is an inflation rate f of 4 % the ideal interest rate will be

  $$\frac{1,08}{1,04} = 0,0386 \approx 0,04 \text{ (which is equal to r-f)}$$

For design calculations making use of estimated figures for the future interest rate as well as for inflation rate instead of the true interest rate r the ideal interest rate of i = r - f may be used.
Substituting of i in all mentioned formulae instead of r means that in an easy and acceptable way a rather big part of the future price developments are integrated.

THE MOST ECONOMIC DESIGN

1. Introduction

The most economic design is defined as that technical acceptable alternative which implies the lowest real costprice for drinking water.

The continued investigation for the most economic diameters for pipelines seems to be verry attractive but there are some problems: f o r instance:

- Alternative investments of long duration can be judged on their merits only within the frame of an all-embracing plan for a longer period.
  There is no independant relation between all different components of a water supply system.
- There are uncertainities regarding the prognosed design criteria as there are water consumption figures, peakfactors, leak-percentages.
- The rather unpredictable variation in the water consumption results in a water demand pattern which influences the optimal operation of the system and thus the variable costs.
- There is a permanent need to evaluate the already predicted interest rates, energy prices, inflation rates and even manpower costs, although changes in the constructed network in practice are impossible anymore.
- Optimalisation is a theoretical method.
  The practice will formulate the boundary conditions (for instance a minimal period for construction purposes is required).

Considering the flexibility in the operation of systems it seems to be practical to make use of the most flexible component of the system viz the (booster) pumps.
In order to overcome whether the in general long term variation (transmission lines) or the short term variation (distribution mains) in both situations there will be installed a serial of pumps.
The complete range may be installed for a relative short period (for instance 5 years). The operational bring into action of the most appropriate combination of pumps depends on the pipeline characteristic and the operating point of the considered pump combination (pump characteristic).
A combination of some well selected centrifugal pumps and one additional small variable rotating pump in general will give sufficient operating flexibility.

However the possibility to make use in the described way of the calculated most economic diameter in practice is greater for transmission lines than for distribution mains.

Summarized it can be concluded that most economical diameters are to be calculated for transmission lines, which are defined from one point to another. Regarding distribution mains, which are commonly integrated within a distribution network it is only possible to compare complete alternative solutions with their costs.

2. Period of analysis

For the comparing of alternatives it is very important that the period of analysis will be the same for all parts components.
Although the most economic design period for distinct components parts of a water supply system may be different a choice of a period of analysis to be equal to the design period of the considered component of the system (transmission line, distribution network) seems to be very attractive.

The important factors that influence the most economic design period were:
- interest rate
- inflation rate
- energy prices
- increase rate of the water demand
- economy of scale.

Regarding the economy of scale an additional remark is to be made:
with sufficient collected relevant cost price data of a water supply company or a lot of comparable familiar companies within a whole region it is possible to find relations for the investment costs of specific components, of water supply systems as there are: water treatment plants
pumping stations
reservoirs
transmission lines
distribution mains.

For instance: for concrete reservoirs:

$FC = k \times Q^E$ with FC = first costs in dfl
E = scale factor (0.6 - 0.7)
Q = required contents in m^3
k = constant value depending on local conditions (5000 - 10000)

For transmission lines:

$FC = a \times D$ with FC = first costs in dfl/m^1
D = diameter in m
a = constant value depending on local conditions (400 - 500)

Other relations sometimes are more convinient:

$FC = b + cD^d$

3. Calculation of the most economical diameter

With a constant water demand

The costs to be compared are first costs and operation and maintenance costs.

The Present Value of all costs excluding the energy costs:

$$PV = FC \tag{1}$$

Regarding the required energy the following considerations have to be mentioned.
To bring up 1 m^3 1 m.w.c. one needs 0.00272 kWh.
With a flow rate 1 m^3/h this implies per year:
0.00272 * 8760 = 23.8 kWh.

If there will be a constant flow rate of Q m^3/h the annual energy costs per m length of pipe P_{ec} will be:

$$P_{ec} = Q * 23{,}8 * \frac{e}{\eta} * \frac{\Delta H}{L} \qquad (2)$$

with e = energy price in dfl/kWh

η = efficiency rate

$\frac{\Delta H}{L}$ = pressure gradient in m/m

Further $\frac{\Delta H}{L} = 0.0826.\lambda.\frac{Q^2}{D^2}$ (formula of Darcy-Weisbach) (3)

Note: here Q in m^3/s.

With the use of equations (2) and (3) and λ = 0.010 one gets:

$$P_{ec} = 32.5 * 10^{-10} \frac{e}{\eta} \frac{Q^3}{D^5}$$

The total annual costs are:

$$P_T = r.(FC) + P_{ec}$$

with FC = a D this means

$$P_T = r.a.D + 32.5 * 10^{-10} * \frac{e}{\eta} * \frac{Q^3}{D^5}$$

This function is minimal $\frac{\partial PT}{\partial D} = 0$

which results in

$$D = 0.05 * \sqrt{Q} \times \sqrt[6]{\frac{e}{\eta \times r \times a}}$$

and with $Q = v \times 1/4\pi D^2$

v = velocity in m/sec.

$$v = 0.14 \times \sqrt{\frac{\eta \times r \times a}{e}}$$

Note: $v = v_{optimal}$ (at the begin of the design period)

So with a constant Q there are four parameters which influence the optimal velocity within the pipeline.

For instance:

η	=	0.4	0.4	0.4	0.4	(-)
r	=	8	8	10	10	(%)
a	=	500	500	500	700	(-)
e	=	0.20	0.40	0.20	0.20	(dfl/kWh)
V_{opt}	=	0.60	0.48	0.65	0.726	(m/s)

Influence of a linear increase of water demand

In this case it is not allowed to look only to the yearly costs. To optimize we use now the present worth of all our costs. Because of the growth of the water demand it is clear that the operation costs increase every year.

First of all we choose some of the parameters
α = linear growth of water demand [%]
β = interest rate [%]
γ = rise of energy costs every year [%]

Again we could say that the total costs are:

$$P_{TC} = P_{FC} + P_{EC} \qquad (1)$$

$$P_{FC} = a * D \qquad (2)$$

$$P_{EC} = 32.5 * 10^{-10} * \frac{e}{\eta\ D^5} * [\ \frac{Q_1^3 * (1 + \gamma)}{(1 + \beta)} + \ldots\ldots + \frac{Q_n^3 * (1 + \gamma)^n}{(1 + \beta)^n}] \qquad (3)$$

We know that $Q_n = (1 + n\alpha) * Q_o$.

If we substitute this in formula (3), the result is:

$$P_{EC} = 32.5 * 10^{-10} * \frac{e * Q_o^3}{\eta * D^5} *$$

$$\underbrace{\left[\frac{(1 + \gamma)\ (1 + \alpha)^3}{1 + \beta} + \ldots\ldots + \frac{(1 + \gamma)^n * (1 + n\alpha)^3}{(1 + \beta\ )^n}\right]}_{\delta^1} \qquad (4)$$

(For δ^1 you find at the end of the appendix a number tables with values for different values of α, β, γ and the length of the design period.)

It is also allowed to approximate ∂^1 to:

$$\delta^1 = \sum_{n=1}^{n} \frac{(1 + n)^3}{(1 + \beta^1)^n} \qquad (5)$$

with $\beta^1 = 0.9 \times (\beta - \gamma$
(only for values of β and $\gamma < 0.15$)

The total costs are now:

$$P_{TC} = a.D + 32.5.10^{-10} * \frac{e * Q_o^3}{\eta * D^5} * \delta^1 \qquad (6)$$

This function is minimal for: $\frac{\partial P_{TC}}{\partial D} = 0$ (7)

After some calculations the result is:

$$V_{opt} = 0.14 * \sqrt[3]{\frac{a * \eta}{\delta^1 * e}} \qquad (8)$$

Influence of an exponantial growth of waterdemand

The begin is quite the same as in the pregoing chapter. Only the growth of the waterdemand is different so the formula for the present worth of the energy costs is:

$$P_{ec} = 32.5 * 10^{-10} * \frac{e * Q_o^3}{\eta * D^5} \left[\frac{1 + \alpha}{1 + \beta'} + \ldots . \frac{(1 + \alpha)^n}{(1 + \beta^1)^n}\right]$$

It is possible to approximate δ^1 through:

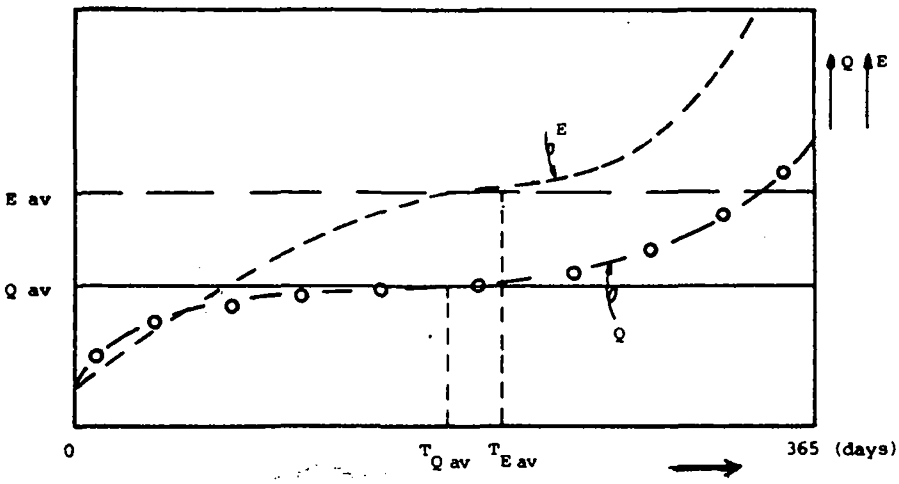
$$\delta^1 = \frac{(1 + 3\alpha - \beta)^{n + 1} - (1 + 3\alpha - \beta^1)}{3\alpha - \beta^1}$$

For this case is the optimal velocity now:

$$V_o = 0.14 * \sqrt[3]{\frac{a * \eta}{\delta^1 * e}}$$

This is the same formula as in the pregoing chapter. The only difference is another δ^1.

Influence of the variation of the waterdemand during a year



Duration curve

During a year there is a large variation of the water demand (take a look at the duration curve above).
It is clear that at we have to produce $Q_{average}$ one day, the need of enerqy is less then $E_{average}$ because of the relation: $E = f\ (Q^3)$.

The average need of energy one uses with a production of $Q_{equivalent}$ is:

$$Q_{equivalent} = \sqrt[3]{\frac{1}{T} \int_0^T Q^3 dt}$$

Calculations of $Q_{equivalent}$ leads to a size of 1.2 till 1.3 times $Q_{average}$. Say there's found in a particular situation 1.26.
The total energy costs are now 8760 times the energy need with a production of 1.26 x $Q_{average}$.
The conclusion is that using $Q_{average}$ (only) for the calculation of the energy costs is not correct.

When one uses 1.26 * $Q_{average}$ the result is:

$$V_o = 0.11 * \sqrt[3]{\frac{a * \eta}{\delta^1 * e}}$$

Example

Situation

A 5 km B

$Q_o \rightarrow$ ———————————————————— $\rightarrow Q_o$

Datalist

Linear growth of water demand : 2 %

Interest rate : 10 %

Rise of energy costs : 6 %

Q_o = 10.000 m^3/day

$P_{F.C}$ = 500 x D

D = diameter (m)

η = 0,6

e = 0,20 dfl/kWh

Question

What is the optimal design for water transport with a design period of 30 years?

$$V_o = 0{,}11 * \sqrt[3]{\frac{a * \eta}{\delta^1 * e}}$$

α = 2 %

β = 10 % See table A-7: δ^1 = 37,1

= 6 %

$$V_o = 0{,}11 * \sqrt[3]{\frac{500 \times 0{,}6}{37{,}1 \times 0{,}20}} = 0{,}37 \text{ m/sec}$$

$$Q_o = 1/4\ \pi\ D^2 \times v_o$$

$$D = \frac{Q_o}{1/4 \pi v_o} = 0{,}631 \text{ m}$$

Answer

The optimal design is a pipe with a diameter of Ø 600 mm (however perhaps a Ø 700 mm may be chosen).

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 2 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.3	11.1	17.5	24.4	31.8	39.7	48.2	57.3
2	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
3	5.6	12.5	20.6	30.5	42.1	55.9	72.1	91.2
4	5.8	13.2	22.5	34.2	48.7	66.8	89.2	116.7
5	6.0	13.9	24.5	38.4	56.5	80.2	110.9	150.5
6	6.2	14.8	26.7	43.2	65.8	96.7	138.7	195.6
7	6.3	15.6	29.1	48.7	76.8	117.0	174.4	255.7
8	6.5	16.6	31.8	55.0	89.8	142.1	220.2	336.2
9	6.7	17.5	34.8	62.1	105.3	173.1	279.0	444.0
10	6.9	18.6	38.0	70.3	123.7	211.4	354.8	588.6
11	7.1	19.7	41.6	79.7	145.5	258.7	452.5	782.7
12	7.3	20.8	45.5	90.4	171.4	317.2	578.3	1043.6
13	7.6	22.1	49.9	102.6	202.3	389.7	740.4	1394.1
14	7.8	23.4	54.6	116.6	238.9	479.3	949.4	1865.2
15	8.0	24.8	59.9	132.5	282.4	590.2	1218.9	2498.1
16	8.2	26.3	65.6	150.8	334.2	727.3	1566.1	3348.1
17	8.5	27.9	71.9	171.6	395.6	896.9	2013.3	4489.0
18	8.7	29.5	78.9	195.4	468.6	1106.6	2589.0	6019.4
19	9.0	31.3	86.6	222.6	555.3	1365.6	3329.7	8070.6
20	9.2	33.2	95.0	253.6	658.1	1685.4	4282.0	9999.0

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 4 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.0	10.0	14.9	19.8	24.5	29.1	33.6	37.9
2	5.2	10.5	16.1	21.9	27.9	34.0	40.2	46.5
3	5.3	11.1	17.5	24.4	31.9	39.9	48.4	57.5
4	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
5	5.6	12.4	20.6	30.4	42.0	55.7	71.9	90.8
6	5.8	13.1	22.4	34.0	48.4	66.3	88.4	115.6
7	6.0	13.9	24.4	38.1	56.1	79.4	109.5	148.3
8	6.1	14.7	26.5	42.8	65.0	95.3	136.3	191.6
9	6.3	15.5	28.9	48.1	75.7	114.9	170.5	249.2
10	6.5	16.4	31.5	54.2	88.2	138.9	214.3	325.7
11	6.7	17.4	34.4	61.1	103.1	168.5	270.2	427.6
12	6.9	18.4	37.5	69.0	120.6	204.9	341.9	563.5
13	7.1	19.5	40.9	78.0	141.4	249.8	433.7	744.9
14	7.3	20.6	44.7	88.2	166.1	305.0	551.5	987.3
15	7.5	21.8	48.9	99.9	195.3	373.0	702.6	1311.1
16	7.7	23.1	53.5	113.2	229.9	456.9	896.4	1743.8
17	7.9	24.4	58.5	128.3	270.9	560.2	1145.0	2321.9
18	8.2	25.9	64.0	145.6	319.4	687.5	1463.8	3094.1
19	8.4	27.4	70.1	165.3	376.8	844.2	1872.5	4124.8
20	8.6	29.0	76.7	187.7	444.8	1037.3	2396.1	5499.9

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 6 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.7	9.0	12.9	16.3	19.4	22.1	24.5	26.6
2	4.9	9.5	13.9	18.0	21.8	25.4	28.7	31.7
3	5.0	10.0	15.0	19.9	24.7	29.4	33.9	38.4
4	5.2	10.6	16.2	22.0	28.0	34.2	40.5	46.8
5	5.3	11.2	17.5	24.5	31.9	40.0	48.6	57.7
6	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
7	5.6	12.4	20.6	30.3	41.9	55.5	71.6	90.4
8	5.8	13.1	22.3	33.9	48.2	65.9	87.7	114.5
9	6.0	13.9	24.3	37.9	55.6	78.6	108.2	146.2
10	6.1	14.6	26.4	42.4	64.3	94.0	134.1	187.9
11	6.3	15.5	28.7	47.6	74.6	112.9	166.9	243.0
12	6.5	16.3	31.2	53.5	86.7	136.0	208.0	315.9
13	6.7	17.3	34.0	60.1	100.9	164.3	262.0	412.4
14	6.9	18.2	37.0	67.7	117.8	198.9	329.9	540.4
15	7.1	19.3	40.3	76.4	137.7	241.5	416.5	710.3
16	7.3	20.4	44.0	86.2	161.1	293.7	527.0	936.0
17	7.5	21.6	48.0	97.3	188.8	357.7	668.1	1236.0
18	7.7	22.8	52.4	110.0	221.5	436.3	848.2	1634.6
19	7.9	24.1	57.2	124.4	260.2	532.8	1078.2	2164.3
20	8.1	25.5	62.5	140.8	305.7	651.2	1371.7	2867.9

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 8 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.5	8.2	11.2	13.6	15.6	17.3	18.5	19.6
2	4.6	8.6	12.0	15.0	17.5	19.6	21.4	22.9
3	4.7	9.1	12.9	16.4	19.6	22.4	24.8	27.0
4	4.9	9.5	14.0	18.1	22.0	25.7	29.1	32.2
5	5.0	10.0	15.0	20.0	24.9	29.6	34.3	38.8
6	5.2	10.6	16.2	22.1	28.2	34.4	40.7	47.2
7	5.3	11.2	17.6	24.5	32.0	40.1	48.8	58.0
8	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
9	5.6	12.4	20.6	30.3	41.8	55.4	71.3	90.0
10	5.8	13.1	22.3	33.7	47.9	65.5	87.1	113.5
11	5.9	13.8	24.1	37.6	55.1	77.8	106.9	144.2
12	6.1	14.6	26.2	42.1	63.6	92.7	131.9	184.4
13	6.3	15.4	28.4	47.1	73.6	111.0	163.6	237.2
14	6.5	16.2	30.9	52.8	85.2	133.2	203.6	306.7
15	6.7	17.1	33.6	59.2	99.8	160.3	254.4	398.3
16	6.8	18.1	36.5	66.5	115.1	193.3	318.0	519.0
17	7.0	19.1	39.8	74.8	134.1	233.8	400.6	678.5
18	7.2	20.2	43.3	84.3	156.5	283.2	504.4	889.2
19	7.4	21.3	47.2	94.9	182.8	343.6	636.5	1167.0
20	7.6	22.5	51.4	107.1	213.8	417.4	804.3	1536.0

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 10 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.2	7.4	9.8	11.6	12.9	13.9	14.6	15.1
2	4.4	7.8	10.5	12.6	14.3	15.5	16.5	17.3
3	4.5	8.2	11.3	13.8	15.9	17.5	18.9	20.0
4	4.6	8.6	12.1	15.1	17.7	19.9	21.7	23.3
5	4.8	9.1	13.0	16.6	19.8	22.6	25.2	27.4
6	4.9	9.6	14.0	18.2	22.2	25.9	29.4	32.6
7	5.0	10.1	15.1	20.1	25.0	29.9	34.6	39.2
8	5.2	10.6	16.3	22.2	28.3	34.6	41.0	47.5
9	5.3	11.2	17.6	24.5	32.1	40.2	48.9	58.2
10	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
11	5.6	12.4	20.5	30.2	41.7	55.2	71.1	89.6
12	5.8	13.1	22.2	33.6	47.7	65.1	86.4	112.5
13	5.9	13.8	24.0	37.4	54.7	77.0	105.7	142.3
14	6.1	14.5	26.0	41.7	62.9	91.6	129.9	181.1
15	6.3	15.3	28.2	46.6	72.6	109.1	160.4	231.0
16	6.4	16.1	30.6	52.1	83.9	130.5	198.8	298.2
17	6.6	17.0	33.2	58.4	97.1	156.5	247.2	385.2
18	6.8	18.0	36.1	65.4	112.6	188.1	308.4	499.3
19	7.0	18.9	39.2	73.4	130.8	226.6	385.8	649.3
20	7.2	20.0	42.6	82.5	152.1	273.4	483.7	846.4

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 12 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.0	6.8	8.7	10.0	10.8	11.4	11.8	12.1
2	4.1	7.1	9.3	10.8	11.9	12.7	13.2	13.6
3	4.3	7.5	9.9	11.8	13.1	14.1	14.9	15.4
4	4.4	7.9	10.6	12.8	14.5	15.8	16.8	17.6
5	4.5	8.3	11.4	14.0	16.1	17.8	19.2	20.3
6	4.6	8.7	12.2	15.3	17.9	20.1	22.1	23.7
7	4.8	9.1	13.1	16.7	20.0	22.9	25.5	27.8
8	4.9	9.6	14.1	18.4	22.4	26.2	29.7	33.0
9	5.0	10.1	15.2	20.2	25.2	30.1	34.9	39.6
10	5.2	10.6	16.3	22.3	28.4	34.7	41.2	47.9
11	5.3	11.2	17.6	24.6	32.2	40.3	49.1	58.4
12	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
13	5.6	12.4	20.5	30.1	41.6	55.0	70.8	89.3
14	5.8	13.0	22.1	33.5	47.5	64.7	85.8	111.6
15	5.9	13.7	23.9	37.2	54.3	76.3	104.6	140.5
16	6.1	14.5	25.9	41.4	62.3	90.4	120.0	178.0
17	6.3	15.2	28.0	46.1	71.7	107.4	157.4	226.7
18	6.4	16.1	30.4	51.5	82.6	128.0	194.2	290.2
19	6.6	16.9	32.9	57.5	95.3	153.0	240.5	373.0
20	6.8	17.8	35.7	64.4	110.2	183.2	298.8	481.1

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 14 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	3.8	6.2	7.7	8.7	9.3	9.6	9.8	10.0
2	3.9	6.5	8.3	9.4	10.1	10.6	10.9	11.1
3	4.0	6.9	8.8	10.1	11.0	11.7	12.1	12.3
4	4.2	7.2	9.4	11.0	12.1	12.9	13.5	13.9
5	4.3	7.6	10.0	11.9	13.3	14.4	15.2	15.7
6	4.4	7.9	10.7	13.0	14.7	16.1	17.1	18.0
7	4.5	8.3	11.5	14.1	16.3	18.1	19.5	20.7
8	4.6	8.7	12.3	15.4	18.1	20.4	22.4	24.1
9	4.8	9.2	13.2	16.9	20.2	23.2	25.9	28.2
10	4.9	9.6	14.2	18.5	22.6	26.4	30.1	33.4
11	5.0	10.1	15.2	20.3	25.3	30.3	35.2	40.0
12	5.2	10.6	16.4	22.4	28.5	34.9	41.5	48.2
13	5.3	11.2	17.6	24.6	32.2	40.4	49.2	58.6
14	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
15	5.6	12.4	20.5	30.1	41.5	54.9	70.6	88.9
16	5.8	13.0	22.1	33.3	47.2	64.3	85.2	110.7
17	5.9	13.7	23.8	37.0	53.9	75.7	103.5	138.8
18	6.1	14.4	25.7	41.1	61.7	89.3	126.2	175.0
19	6.2	15.2	27.8	45.7	70.8	105.8	154.5	221.9
20	6.4	16.0	30.1	50.9	81.3	125.7	189.9	282.7

This Table contents the factor DELTA with linear growth

The linear growth is 1 %
The interest-rate is 16 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	3.6	5.8	7.0	7.6	8.0	8.3	8.4	8.5
2	3.8	6.0	7.4	8.2	8.7	9.0	9.2	9.3
3	3.9	6.3	7.9	8.8	9.4	9.8	10.1	10.2
4	4.0	6.6	8.4	9.5	10.3	10.8	11.1	11.3
5	4.1	6.9	8.9	10.3	11.2	11.9	12.3	12.6
6	4.2	7.3	9.5	11.1	12.3	13.1	13.7	14.2
7	4.3	7.6	10.1	12.1	13.5	14.6	15.4	16.1
8	4.4	8.0	10.8	13.1	14.9	16.3	17.4	18.3
9	4.5	8.4	11.6	14.3	16.5	18.3	19.8	21.1
10	4.7	8.8	12.4	15.6	18.3	20.7	22.7	24.5
11	4.8	9.2	13.3	17.0	20.4	23.4	26.2	28.6
12	4.9	9.7	14.2	18.6	22.8	26.7	30.4	33.8
13	5.0	10.2	15.3	20.4	25.5	30.5	35.5	40.3
14	5.2	10.7	16.4	22.4	28.7	35.1	41.7	48.5
15	5.3	11.2	17.6	24.7	32.3	40.5	49.4	58.8
16	5.5	11.8	19.0	27.2	36.5	47.0	58.8	71.9
17	5.6	12.4	20.4	30.0	41.4	54.7	70.4	88.6
18	5.8	13.0	22.0	33.2	47.0	64.0	84.7	109.9
19	5.9	13.7	23.7	36.8	53.6	75.0	102.4	137.2
20	6.1	14.4	25.6	40.8	61.1	88.3	124.4	172.2

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 2 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.8	13.0	21.8	32.4	44.8	59.2	75.7	94.5
2	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
3	6.2	14.6	26.0	41.0	60.5	85.5	117.1	156.7
4	6.3	15.5	28.4	46.3	70.7	103.5	147.0	204.2
5	6.5	16.4	31.1	52.4	82.8	125.7	185.5	267.8
6	6.7	17.4	34.0	59.3	97.2	153.3	235.1	353.2
7	6.9	18.5	37.3	67.3	114.4	187.4	299.1	460.1
8	7.2	19.6	40.9	76.4	134.9	229.7	381.8	622.9
9	7.4	20.8	44.8	86.8	159.2	282.3	488.8	831.6
10	7.6	22.1	49.2	98.8	188.3	347.5	627.2	1113.2
11	7.8	23.4	54.0	112.5	223.0	428.5	806.3	1493.2
12	8.1	24.9	59.3	128.2	264.3	529.2	1038.2	2006.2
13	8.3	26.4	65.1	146.2	313.6	654.1	1338.3	2698.4
14	8.5	28.0	71.5	166.9	372.4	809.2	1726.6	3632.3
15	8.8	29.8	78.6	190.5	442.4	1001.8	2228.8	4891.4
16	9.1	31.6	86.5	217.6	525.8	1240.7	2878.0	6587.9
17	9.3	33.5	95.1	248.5	625.2	1536.9	3716.6	8871.9
18	9.6	35.6	104.5	284.0	743.5	1904.0	4799.1	9999.0
19	9.9	37.8	115.0	324.5	884.2	2358.7	6195.3	9999.0
20	10.2	40.1	126.5	370.9	1051.6	2921.4	7994.5	9999.0

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 4 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.5	11.6	18.5	25.9	33.8	42.2	50.9	59.8
2	5.6	12.3	20.1	28.9	38.9	49.9	62.0	75.0
3	5.8	13.0	21.8	32.4	44.9	59.4	76.0	94.9
4	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
5	6.1	14.6	26.0	40.9	60.4	85.2	116.6	155.9
6	6.3	15.5	28.3	46.1	70.3	102.7	145.7	202.1
7	6.5	16.4	30.9	52.0	82.1	124.3	183.0	263.6
8	6.7	17.4	33.8	58.8	96.0	150.9	230.8	345.7
9	6.9	18.4	37.0	66.5	112.6	183.8	292.2	455.6
10	7.1	19.5	40.4	75.3	132.3	224.4	371.1	602.6
11	7.3	20.7	44.3	85.3	155.7	274.5	472.7	799.8
12	7.6	21.9	48.5	96.9	183.5	336.5	603.5	1064.2
13	7.8	23.2	53.1	110.0	216.5	413.2	771.9	1419.1
14	8.0	24.6	58.2	125.0	255.8	508.1	988.9	1895.2
15	8.2	26.1	63.8	142.2	302.5	625.4	1268.2	2534.2
16	8.5	27.6	70.0	161.9	357.9	770.4	1627.9	3391.3
17	8.7	29.3	76.8	184.3	423.7	949.7	2090.9	4540.5
18	9.0	31.1	84.3	209.9	501.9	1171.2	2686.5	6080.4
19	9.2	33.0	92.5	239.2	594.7	1444.8	3452.3	8142.1
20	9.5	34.9	101.5	272.6	704.9	1782.5	4436.2	9999.0

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 6 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.1	10.4	15.8	21.0	26.2	31.1	35.7	40.1
2	5.3	11.0	17.1	23.4	29.8	36.3	42.7	49.0
3	5.5	11.7	18.5	26.0	34.1	42.6	51.4	60.6
4	5.6	12.3	20.1	29.1	39.1	50.2	62.4	75.6
5	5.8	13.0	21.9	32.5	45.0	59.6	76.3	95.3
6	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
7	6.1	14.6	25.9	40.8	60.2	84.9	116.2	155.2
8	6.3	15.4	28.2	45.9	69.9	102.0	144.5	200.1
9	6.5	16.3	30.8	51.7	81.3	123.0	180.6	259.6
10	6.7	17.3	33.6	58.2	94.9	148.7	226.0	338.7
11	6.9	18.3	36.6	65.7	110.9	180.4	285.7	443.0
12	7.1	19.4	40.0	74.2	129.9	219.3	361.2	583.7
13	7.3	20.5	43.7	83.9	152.4	267.3	457.8	770.3
14	7.5	21.7	47.0	95.0	179.0	326.3	581.6	1019.2
15	7.7	23.0	52.3	107.6	210.5	399.0	740.2	1351.2
16	8.0	24.3	57.2	122.0	247.9	488.6	943.6	1794.4
17	8.2	25.8	62.6	138.5	292.1	598.9	1204.3	2385.7
18	8.4	27.3	68.6	157.2	344.5	734.8	1538.4	3174.7
19	8.7	28.9	75.1	178.5	406.5	902.1	1966.3	4226.0
20	8.9	30.6	82.2	202.8	479.9	1108.1	2514.3	5629.0

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 8 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.9	9.4	13.6	17.4	20.7	23.6	26.1	28.3
2	5.0	9.9	14.7	19.2	23.4	27.2	30.7	33.8
3	5.2	10.5	15.9	21.2	26.5	31.5	36.3	40.8
4	5.3	11.1	17.2	23.6	30.1	36.7	43.3	49.8
5	5.5	11.7	18.6	26.2	34.3	43.0	52.0	61.3
6	5.6	12.4	20.2	29.2	39.3	50.6	62.9	76.3
7	5.8	13.0	21.9	32.6	45.1	59.8	76.6	95.7
8	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
9	6.1	14.6	25.9	40.7	60.0	84.6	115.7	154.4
10	6.3	15.4	28.1	45.7	69.5	101.3	143.3	198.2
11	6.5	16.3	30.6	51.3	80.7	121.7	178.4	255.9
12	6.7	17.2	33.4	57.7	93.8	146.6	222.9	332.0
13	6.9	18.2	36.3	65.0	109.3	177.2	279.6	432.7
14	7.1	19.2	39.6	73.2	127.6	214.6	351.0	566.1
15	7.3	20.3	43.2	82.6	149.2	260.5	443.9	743.0
16	7.5	21.5	47.2	93.3	174.8	316.8	561.3	977.6
17	7.7	22.8	51.5	105.4	204.9	385.8	711.0	1289.0
18	7.9	24.1	56.3	119.2	240.5	470.6	902.1	1702.4
19	8.1	25.5	61.5	134.9	282.5	574.6	1145.8	2251.1
20	8.4	26.9	67.2	152.8	332.1	702.1	1456.8	2979.2

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 10 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.6	8.6	11.9	14.6	16.7	18.5	19.9	20.9
2	4.7	9.0	12.8	16.0	18.7	21.0	22.9	24.5
3	4.9	9.5	13.7	17.6	21.0	24.0	26.7	28.9
4	5.0	10.0	14.8	19.4	23.7	27.6	31.2	34.5
5	5.2	10.5	16.0	21.4	26.8	31.9	36.9	41.5
6	5.3	11.1	17.3	23.0	30.4	37.1	43.8	50.5
7	5.5	11.7	18.7	26.4	34.6	43.3	52.5	62.0
8	5.6	12.4	20.3	29.3	39.5	50.9	63.3	76.9
9	5.8	13.1	22.0	32.6	45.2	60.0	76.9	96.1
10	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
11	6.1	14.6	25.8	40.7	59.9	84.4	115.2	153.7
12	6.3	15.4	28.1	45.5	69.1	100.6	142.2	196.4
13	6.5	16.2	30.5	51.0	80.0	120.5	176.3	252.3
14	6.7	17.1	33.1	57.2	92.8	144.6	219.3	325.7
15	6.9	18.1	36.1	64.3	107.8	174.1	273.9	422.4
16	7.1	19.1	39.3	72.3	125.5	210.1	343.1	549.7
17	7.3	20.2	42.8	81.3	146.3	254.1	430.9	717.6
18	7.5	21.3	46.6	91.6	170.8	307.9	542.4	939.2
19	7.7	22.5	50.8	103.3	199.6	373.5	683.9	1231.0
20	7.9	23.8	55.4	116.6	233.5	453.9	863.8	1618.3

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 12 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.4	7.8	10.4	12.4	13.8	14.9	15.6	16.2
2	4.5	8.2	11.2	13.5	15.3	16.7	17.8	18.5
3	4.6	8.6	12.0	14.8	17.0	18.9	20.3	21.4
4	4.8	9.1	12.9	16.2	19.0	21.4	23.4	25.0
5	4.9	9.6	13.9	17.8	21.3	24.5	27.2	29.5
6	5.0	10.1	14.9	19.6	24.0	28.1	31.8	35.1
7	5.2	10.6	16.1	21.6	27.1	32.4	37.4	42.2
8	5.3	11.2	17.4	23.9	30.7	37.5	44.4	51.2
9	5.5	11.8	18.8	26.5	34.8	43.7	53.0	62.8
10	5.6	12.4	20.3	29.4	39.7	51.2	63.8	77.5
11	5.8	13.1	22.0	32.7	45.4	60.1	77.1	96.5
12	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
13	6.1	14.5	25.8	40.6	59.7	84.1	114.8	153.1
14	6.3	15.3	28.0	45.3	68.8	100.0	141.1	194.7
15	6.5	16.2	30.3	50.7	79.4	119.3	174.2	248.9
16	6.7	17.1	32.9	56.7	91.8	142.8	215.9	319.8
17	6.8	18.0	35.8	63.6	106.4	171.2	268.5	412.6
18	7.0	19.0	38.9	71.4	123.5	205.9	334.9	534.3
19	7.2	20.1	42.3	80.1	143.5	240.1	410.7	694.0
20	7.4	21.2	46.0	90.1	167.0	299.5	524.8	903.7

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 14 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.2	7.1	9.2	10.6	11.6	12.3	12.7	13.0
2	4.3	7.5	9.9	11.6	12.0	13.6	14.2	14.6
3	4.4	7.9	10.5	12.6	14.1	15.2	16.0	16.6
4	4.5	8.3	11.3	13.7	15.6	17.1	18.2	19.0
5	4.7	8.7	12.1	15.0	17.3	19.2	20.8	22.0
6	4.8	9.1	13.0	16.4	19.3	21.8	23.9	25.6
7	4.9	9.6	14.0	18.0	21.6	24.9	27.7	30.1
8	5.1	10.1	15.1	19.8	24.3	28.5	32.3	35.8
9	5.2	10.6	16.2	21.8	27.4	32.8	38.0	42.9
10	5.3	11.2	17.5	24.1	30.9	37.9	45.0	52.0
11	5.5	11.8	18.9	26.6	35.1	44.1	53.5	63.4
12	5.6	12.4	20.4	29.5	39.9	51.4	64.2	78.1
13	5.8	13.1	22.0	32.8	45.5	60.3	77.4	96.9
14	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
15	6.1	14.5	25.8	40.5	59.6	83.0	114.4	152.5
16	6.3	15.3	27.9	45.1	68.4	99.4	140.1	193.0
17	6.5	16.1	30.2	50.4	78.0	118.2	172.3	245.7
18	6.7	17.0	32.8	56.3	90.9	141.0	212.7	314.2
19	6.8	17.9	35.5	63.0	105.0	168.5	263.4	403.4
20	7.0	18.9	38.6	70.5	121.5	201.9	327.1	519.9

This Table contents the factor DELTA with linear growth

The linear growth is 2 %
The interest-rate is 16 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.0	6.5	8.2	9.3	9.9	10.3	10.5	10.7
2	4.1	6.9	8.8	10.0	10.8	11.3	11.7	11.9
3	4.2	7.2	9.3	10.8	11.9	12.5	13.0	13.3
4	4.3	7.6	10.0	11.8	13.0	13.9	14.5	15.0
5	4.4	7.9	10.7	12.8	14.4	15.5	16.4	17.0
6	4.5	8.3	11.4	13.9	15.9	17.4	18.6	19.5
7	4.7	8.8	12.3	15.2	17.6	19.6	21.2	22.5
8	4.8	9.2	13.2	16.6	19.6	22.2	24.4	26.2
9	4.9	9.7	14.1	18.2	21.9	25.3	28.2	30.8
10	5.1	10.2	15.2	20.0	24.6	28.9	32.9	36.5
11	5.2	10.7	16.3	22.0	27.7	33.2	38.5	43.6
12	5.4	11.2	17.6	24.3	31.2	38.3	45.5	52.7
13	5.5	11.8	18.9	26.8	35.3	44.4	54.0	64.1
14	5.7	12.4	20.4	29.6	40.1	51.7	64.6	78.7
15	5.8	13.1	22.0	32.8	45.6	60.5	77.7	97.3
16	6.0	13.8	23.8	36.4	52.0	71.0	93.9	121.1
17	6.1	14.5	25.7	40.4	59.4	83.6	114.0	151.8
18	6.3	15.3	27.8	45.0	68.1	98.0	139.1	191.4
19	6.5	16.1	30.1	50.1	78.2	117.1	170.4	242.6
20	6.6	16.9	32.6	55.9	90.0	139.2	209.6	308.0

This Table contents the factor DELTA with linear growth

The linear growth is 3 %
The interest-rate is 2 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	6.3	15.1	27.0	42.3	61.5	85.2	113.7	147.4
2	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
3	6.7	17.1	32.4	54.2	84.5	125.6	180.1	251.2
4	6.9	18.1	35.5	61.5	99.4	153.3	228.3	331.1
5	7.1	19.3	39.0	69.9	117.2	187.7	290.8	438.9
6	7.4	20.5	42.8	79.6	138.5	230.6	371.7	584.3
7	7.6	21.7	47.0	90.7	163.9	283.9	476.6	780.8
8	7.8	23.1	51.7	103.4	194.3	350.3	612.7	1046.7
9	8.1	24.6	56.9	118.1	230.6	432.9	789.3	1406.5
10	8.3	26.1	62.6	134.9	274.1	535.9	1018.6	1893.7
11	8.6	27.7	68.9	154.2	326.0	664.1	1316.3	2553.2
12	8.8	29.5	75.9	176.4	388.1	823.7	1702.6	3446.0
13	9.1	31.3	83.5	201.8	462.2	1022.5	2204.0	4653.9
14	9.4	33.3	92.0	231.0	550.8	1269.7	2854.1	6287.2
15	9.6	35.4	101.4	264.5	656.6	1577.3	3696.8	8494.1
16	9.9	37.6	111.7	302.9	782.9	1959.7	4788.3	9999.0
17	10.2	40.0	123.1	347.0	933.6	2434.7	6200.9	9999.0
18	10.5	42.5	135.7	397.5	1113.2	3024.3	8027.4	9999.0
19	10.8	45.2	149.5	455.4	1327.3	3755.8	9999.0	9999.0
20	11.2	48.0	164.8	521.6	1582.3	4662.4	9999.0	9999.0

This Table contents the factor DELTA with linear growth

The linear growth is 3 %
The interest-rate is 4 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.9	13.5	22.7	33.4	45.7	59.5	74.4	90.6
2	6.1	14.3	24.7	37.6	53.1	71.1	91.9	115.3
3	6.3	15.2	27.0	42.4	61.7	85.5	114.2	148.1
4	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
5	6.7	17.0	32.3	54.1	84.3	125.1	179.2	249.9
6	6.9	18.1	35.4	61.2	98.8	152.1	226.2	327.6
7	7.1	19.2	38.8	69.4	116.1	185.5	286.7	431.7
8	7.3	20.4	42.5	78.8	136.7	226.9	364.7	571.5
9	7.6	21.6	46.6	89.6	161.3	278.2	465.3	759.3
10	7.8	22.9	51.2	101.9	190.5	341.8	595.1	1011.8
11	8.0	24.4	56.2	116.0	225.4	420.7	762.7	1351.5
12	8.3	25.9	61.7	132.1	266.9	518.5	979.3	1808.8
13	8.5	27.5	67.8	150.7	316.3	639.8	1259.0	2424.3
14	8.8	29.1	74.5	171.8	375.3	790.2	1620.3	3252.7
15	9.0	30.9	81.9	196.1	445.4	976.7	2086.8	4367.1
16	9.3	32.9	90.0	223.9	529.0	1207.8	2688.8	5865.5
17	9.6	34.9	99.0	255.7	628.4	1494.0	3465.3	7878.8
18	9.9	37.0	108.8	292.1	746.7	1848.4	4466.2	9999.0
19	10.1	39.3	119.7	333.7	887.4	2286.9	5755.5	9999.0
20	10.4	41.7	131.7	381.2	1054.6	2829.1	7414.7	9999.0

This Table contents the factor DELTA with linear growth

The linear growth is 3 .%
The interest-rate is 6 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.6	12.1	19.2	26.9	34.8	42.8	50.8	58.6
2	5.8	12.8	20.9	30.0	40.0	50.6	61.7	73.0
3	5.9	13.5	22.8	33.7	46.1	60.1	75.3	91.0
4	6.1	14.3	24.8	37.8	53.4	71.6	92.6	116.4
5	6.3	15.2	27.1	42.5	61.9	85.0	114.6	140.0
6	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
7	6.7	17.0	32.3	54.0	84.0	124.6	170.5	240.6
8	6.9	18.0	35.3	61.0	98.2	151.0	224.2	324.2
9	7.1	19.1	38.6	68.9	115.1	183.4	282.9	425.0
10	7.3	20.3	42.2	78.1	135.0	223.5	350.1	559.5
11	7.5	21.5	46.2	88.5	158.8	272.9	454.6	739.1
12	7.8	22.8	50.6	100.4	186.9	333.9	578.7	979.3
13	8.0	24.2	55.5	114.0	220.4	409.3	738.1	1300.6
14	8.2	25.6	60.8	129.6	260.1	502.4	943.0	1730.8
15	8.5	27.2	66.7	147.3	307.3	617.3	1206.4	2306.5
16	8.7	28.8	73.2	167.6	363.3	759.3	1544.9	3077.0
17	9.0	30.6	80.3	190.8	429.8	934.7	1979.9	4107.9
18	9.2	32.4	88.1	217.3	508.8	1151.1	2530.7	5486.4
19	9.5	34.4	96.7	247.5	602.4	1418.1	3256.2	7320.7
20	9.8	36.5	106.1	282.0	713.4	1747.4	4176.7	9780.8

This Table contents the factor DELTA with linear growth

The linear growth is 3 %
The interest-rate is 8 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.3	10.9	16.5	21.9	27.1	31.9	36.2	40.0
2	5.4	11.5	17.8	24.4	30.8	37.1	43.1	48.7
3	5.6	12.1	19.4	27.1	35.2	43.5	51.7	59.8
4	5.8	12.8	21.0	30.3	40.4	51.2	62.6	74.2
5	6.0	13.6	22.9	33.9	46.5	60.6	76.2	93.0
6	6.1	14.4	24.9	37.9	53.6	72.1	93.4	117.4
7	6.3	15.2	27.1	42.6	62.1	86.1	115.1	149.5
8	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
9	6.7	17.0	32.2	53.8	83.8	124.2	177.7	247.4
10	6.9	18.0	35.1	60.7	97.6	149.9	222.3	321.0
11	7.1	19.1	38.4	68.5	114.0	181.5	279.2	418.6
12	7.3	20.2	41.9	77.3	133.4	220.2	351.9	548.1
13	7.5	21.4	45.8	87.5	156.4	267.9	444.7	720.2
14	7.7	22.6	50.1	99.0	183.6	326.5	563.3	949.0
15	8.0	24.0	54.8	112.1	215.7	398.6	715.1	1253.5
16	8.2	25.4	60.0	127.1	253.8	487.3	909.3	1650.9
17	8.4	26.9	65.7	144.2	298.9	596.5	1157.7	2190.5
18	8.7	28.5	71.9	163.7	352.2	730.7	1475.6	2916.9
19	8.9	30.2	78.8	185.8	415.3	895.9	1882.3	3872.9
20	9.2	32.0	86.3	211.1	490.0	1099.0	2402.2	5144.6

This Table contents the factor DELTA with linear growth

The linear growth is 3 %
The interest-rate is 10 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	5.0	9.8	14.2	18.2	21.5	24.4	26.0	28.7
2	5.2	10.4	15.4	20.1	24.3	28.1	31.3	34.1
3	5.3	10.9	16.6	22.2	27.5	32.5	37.0	41.0
4	5.5	11.5	18.0	24.7	31.3	37.8	44.0	49.8
5	5.6	12.2	19.5	27.4	35.7	44.1	52.6	61.0
6	5.8	12.9	21.2	30.5	40.8	51.9	63.5	75.4
7	6.0	13.6	23.0	34.1	46.9	61.2	77.0	94.2
8	6.1	14.4	25.0	38.1	53.9	72.6	94.1	118.5
9	6.3	15.2	27.1	42.7	62.2	86.4	115.5	150.2
10	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
11	6.7	17.0	32.2	53.7	83.5	123.8	177.0	246.3
12	6.9	18.0	35.0	60.4	97.1	140.9	220.5	310.0
13	7.1	19.0	38.2	68.0	113.1	179.6	275.0	412.6
14	7.3	20.1	41.7	76.7	131.9	217.1	346.0	537.4
15	7.5	21.3	45.5	86.5	154.2	263.1	435.2	702.4
16	7.7	22.5	49.6	97.7	180.4	319.5	548.9	920.7
17	7.9	23.8	54.2	110.4	211.3	388.6	693.6	1209.8
18	8.2	25.2	59.2	124.8	247.9	473.2	870.0	1592.5
19	8.4	26.7	64.7	141.3	291.0	577.0	1112.8	2099.3
20	8.6	28.2	70.7	159.9	341.8	704.2	1411.9	2770.6

This Table contents the factor DELTA with linear growth

The linear growth is 3 %
The interest-rate is 12 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.7	8.9	12.4	15.3	17.5	19.2	20.5	21.5
2	4.9	9.4	13.4	16.8	19.6	21.8	23.6	25.0
3	5.0	9.9	14.4	18.4	22.0	24.9	27.4	29.5
4	5.2	10.4	15.6	20.4	24.7	28.7	32.1	35.0
5	5.3	11.0	16.8	22.5	28.0	33.1	37.8	42.0
6	5.5	11.6	18.2	24.9	31.7	38.4	44.0	50.8
7	5.6	12.2	19.7	27.7	36.1	44.8	53.5	62.1
8	5.8	12.9	21.3	30.8	41.2	52.5	64.3	76.6
9	6.0	13.6	23.1	34.3	47.2	61.8	77.8	95.3
10	6.1	14.4	25.0	38.3	54.2	73.0	94.8	119.5
11	6.3	15.2	27.2	42.7	62.4	86.7	116.0	150.8
12	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
13	6.7	17.0	32.1	53.6	83.3	123.4	176.3	245.1
14	6.9	17.9	34.9	60.2	96.6	147.9	218.8	315.1
15	7.1	18.9	38.0	67.6	112.1	177.8	272.4	406.8
16	7.3	20.0	41.4	76.0	130.5	214.2	340.4	527.3
17	7.5	21.2	45.1	85.6	152.0	258.6	426.4	685.7
18	7.7	22.4	49.2	96.4	177.4	312.9	535.4	894.3
19	7.9	23.6	53.6	108.7	207.2	379.2	673.6	1169.0
20	8.1	25.0	58.5	122.6	242.3	460.1	848.8	1531.0

This Table contents the factor DELTA with linear growth

The linear growth is 3 %
The interest-rate is 14 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.5	8.1	10.9	13.0	14.5	15.5	16.3	16.8
2	4.6	8.5	11.7	14.2	16.1	17.4	18.4	19.2
3	4.8	9.0	12.6	15.5	17.9	19.7	21.1	22.1
4	4.9	9.5	13.5	17.0	20.0	22.3	24.3	25.8
5	5.0	10.0	14.6	18.7	22.4	25.5	28.1	30.3
6	5.2	10.5	15.7	20.6	25.2	29.2	32.8	35.9
7	5.3	11.1	17.0	22.8	28.4	33.7	38.5	43.0
8	5.5	11.7	18.3	25.2	32.2	39.0	45.6	51.9
9	5.7	12.3	19.8	27.9	36.5	45.4	54.4	63.3
10	5.8	13.0	21.4	31.0	41.6	53.1	65.2	77.0
11	6.0	13.7	23.2	34.5	47.5	62.3	78.6	96.4
12	6.2	14.4	25.1	38.4	54.5	73.4	95.4	120.4
13	6.3	15.2	27.2	42.8	62.6	86.9	116.4	151.5
14	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
15	6.7	17.0	32.1	53.5	83.1	123.0	175.7	244.1
16	6.9	17.9	34.8	59.9	96.1	146.9	217.1	312.3
17	7.1	18.9	37.8	67.2	111.3	176.0	269.3	401.3
18	7.3	19.9	41.1	75.4	129.1	211.4	335.1	517.7
19	7.5	21.1	44.8	84.7	150.0	254.4	418.0	670.0
20	7.7	22.2	48.7	95.2	174.5	306.7	522.6	869.6

This Table contents the factor DELTA with linear growth

The linear growth is 3 %
The interest-rate is 16 %
A=Length of design-period [years]
B=Rise of energy-costs [%]

B / A	5	10	15	20	25	30	35	40
1	4.3	7.4	9.7	11.2	12.2	12.8	13.2	13.5
2	4.4	7.8	10.3	12.2	13.4	14.3	14.8	15.2
3	4.5	8.2	11.1	13.2	14.8	15.9	16.7	17.3
4	4.7	8.6	11.9	14.4	16.4	17.9	19.0	19.7
5	4.8	9.1	12.8	15.8	18.2	20.2	21.6	22.8
6	4.9	9.6	13.7	17.3	20.4	22.9	24.9	26.5
7	5.1	10.1	14.8	19.0	22.8	26.0	28.8	31.1
8	5.2	10.6	15.9	20.9	25.6	29.8	33.5	36.8
9	5.4	11.1	17.1	23.1	28.8	34.3	39.3	43.9
10	5.5	11.7	18.5	25.5	32.6	39.6	46.4	53.0
11	5.7	12.4	19.9	28.2	36.9	46.0	55.2	64.4
12	5.8	13.0	21.5	31.2	42.0	53.6	66.0	78.9
13	6.0	13.7	23.3	34.7	47.9	62.8	79.4	97.5
14	6.2	14.5	25.2	38.5	54.7	73.9	96.1	121.4
15	6.3	15.2	27.3	42.9	62.7	87.2	116.8	152.1
16	6.5	16.1	29.5	47.8	72.0	103.2	142.7	191.7
17	6.7	16.9	32.0	53.4	82.9	122.6	175.0	243.1
18	6.9	17.9	34.7	59.7	95.6	146.0	215.5	309.6
19	7.1	18.8	37.7	66.8	110.4	174.4	266.3	396.1
20	7.2	19.9	40.9	74.8	127.8	208.7	330.0	508.6

HYDRAULIC BACKGROUND AND THE APPLICATION OF FLOW FORMULAE

The formula of Darcy-Weisbach to calculate the friction loss in pipe sections is

$$\Delta H_{fr} = 0.0826 \; \lambda \; L \, \frac{Q^2}{D^5}$$

The value of the friction coefficient λ was depending on the type of flow, more specific depending on the grade of turbulence of the flow rate which can be characterized with the Reynolds number (Re)

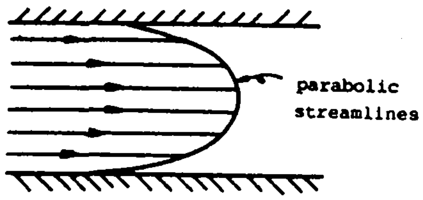
$$Re = \frac{\bar{v} \, D}{\nu}$$

The kinematic viscosity ν varies with the temperature of the water in the pipe
In the following table some numerical values of the kinematic viscosity at different temperatures are presented.

T	ν
0 °C	1.79×10^{-6} m²/s
5 °C	1.52×10^{-6} m²/s
10 °C	1.31×10^{-6} m²/s
15 °C	1.15×10^{-6} m²/s
20 °C	1.01×10^{-6} m²/s
25 °C	0.89×10^{-6} m²/s
30 °C	0.80×10^{-6} m²/s

The following types of flow are possible:

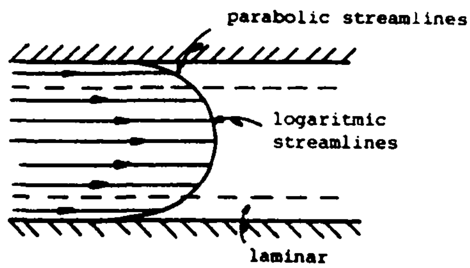
- Laminar flow



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

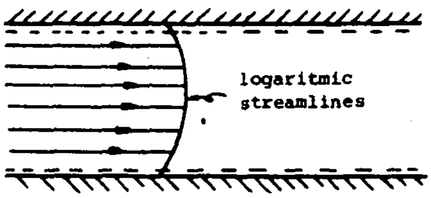
This type of flow will appears when Re < 2100 à 2300.

- Turbulent flow
  - Along a "hydraulic smooth wall"
    Along the wall surface itself there will be always a laminar flow layer.
    The uneveness of the wall is small compared with this laminar layer so after all λ only depends of the kinematic viscosity, the velocity and the diameter (= Reynolds number).



$\frac{1}{\sqrt{\lambda}} = 2 \log Re \sqrt{\lambda} - 0,80$ (Empirical formula)

  - <u>Along a "hydraulic rough wall"</u>
    The laminar layer has become relatively thinner compared with the magnitude of the uneveness, so the total dividing of the streamlines is logaritmic.



$\frac{1}{\sqrt{\lambda}} = (2 \log \frac{D}{k}) + 1.14$ (Empirical formula of Nikuradse)

It is recommended to use for the turbulent flow whatever the flow is along a hydraulic rough or hydraulic smooth wall the formula of Colebrook.

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

This formula tends to the hydraulic smooth formula when D is large ($\frac{k}{3.7\ D}$ is very small) and when Re is high ($\frac{2.51}{Re\sqrt{\lambda}}$ is very small) it tends to the hydraulic rough formula.

The value λ is further depending on the size and the shape of the (relative) wall roughness, characterized by $\frac{D}{k}$

with: D = diameter of the pipe in mm
k = wall roughness coefficient in mm

All pregoing considerations are combined in one graph: the Moody diagram, which give the relation between the friction coefficient λ, Reynolds number Re and the wall roughness characteristic $\frac{D}{k}$.

After interpreting this diagram from the relevant flow characteristic graph the pressure gradient may be found (c.f. figure 1).

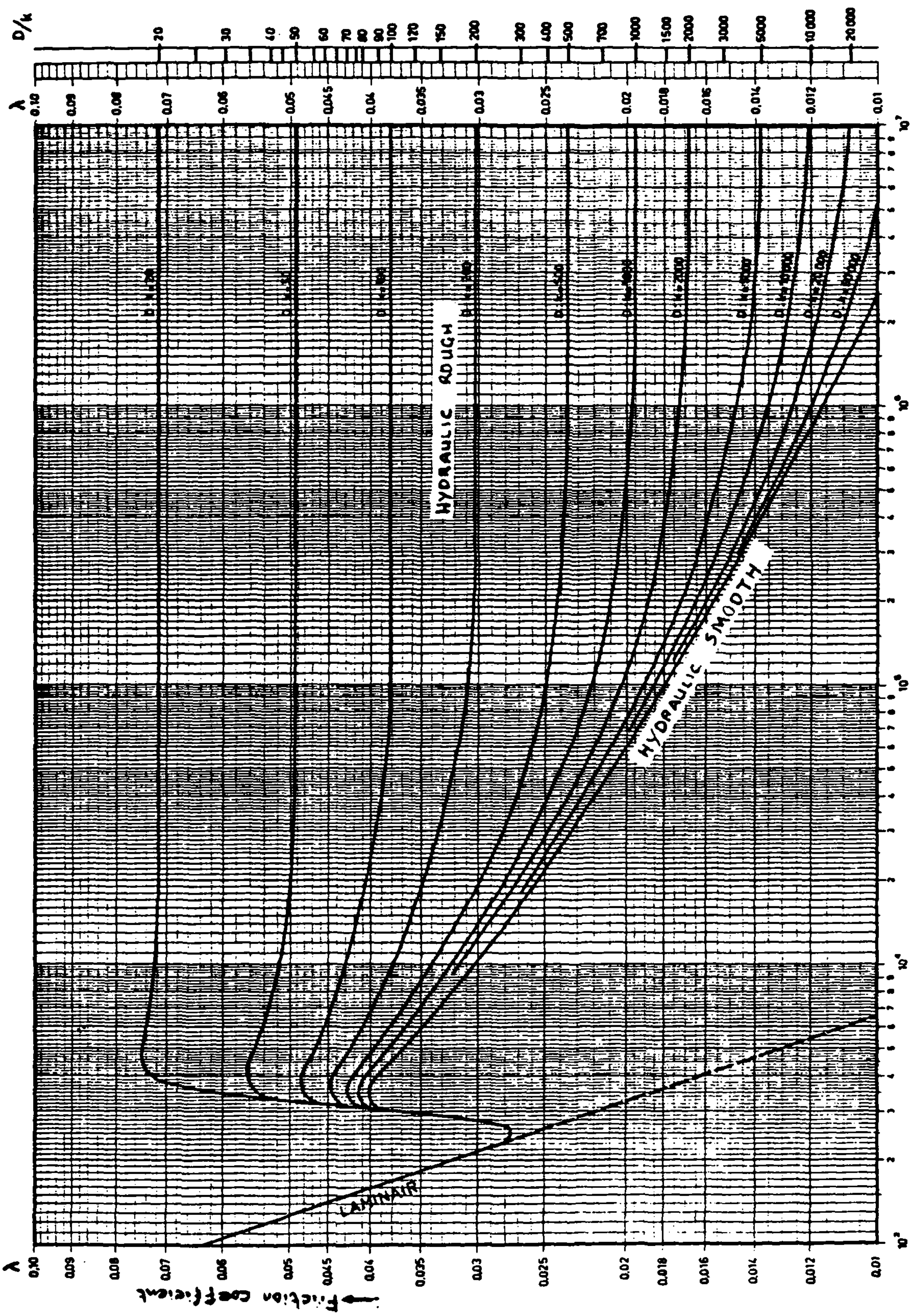


Fig. 1 Relation frictioncoefficient - Reynolds number and wall roughness (Colebrook)

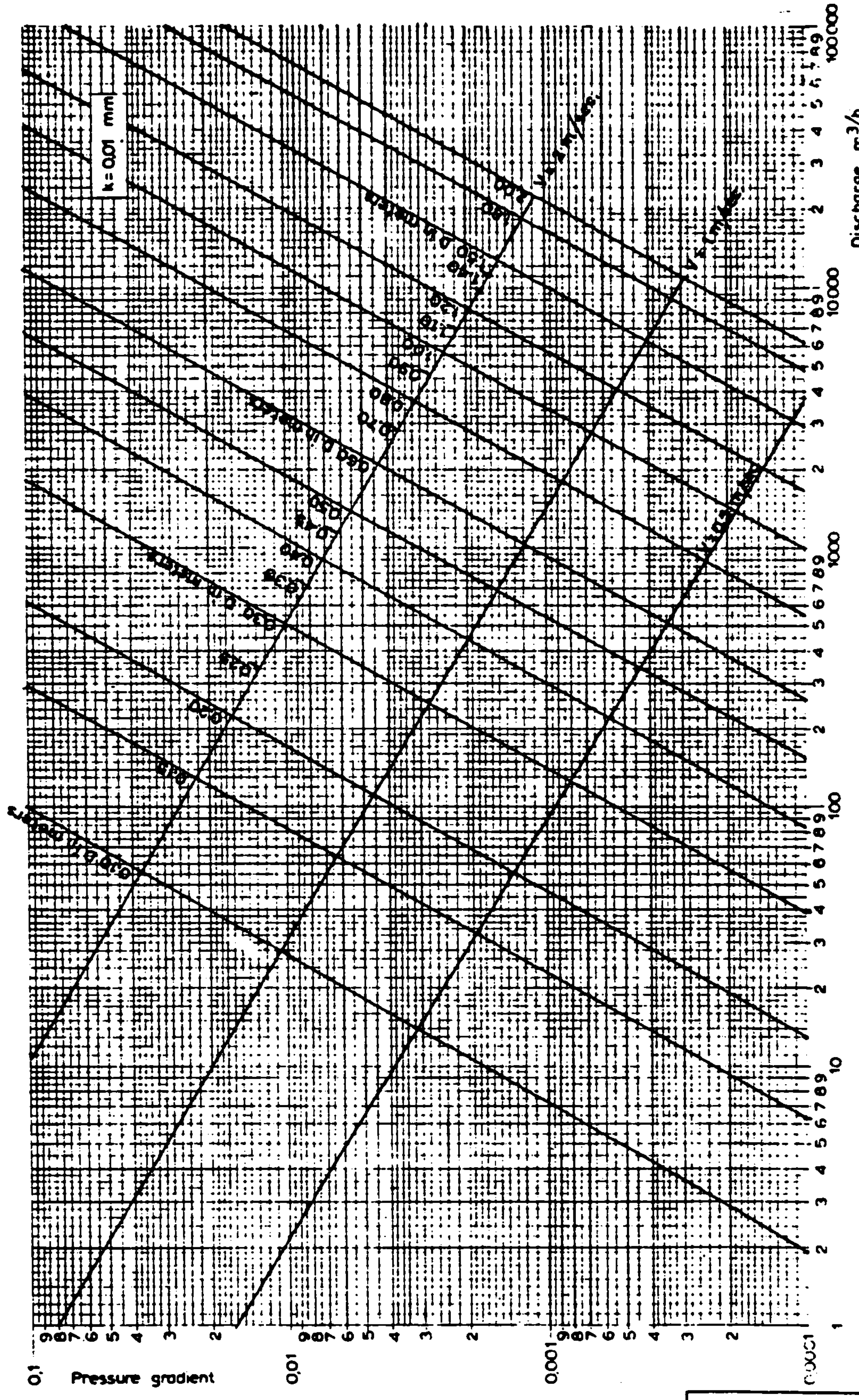


FIG. 2a FLOW CHARACTERISTICS FOR PIPES
(temp. 0°C wallroughness) k=0,01 mm.

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WATER DISTRIBUTION

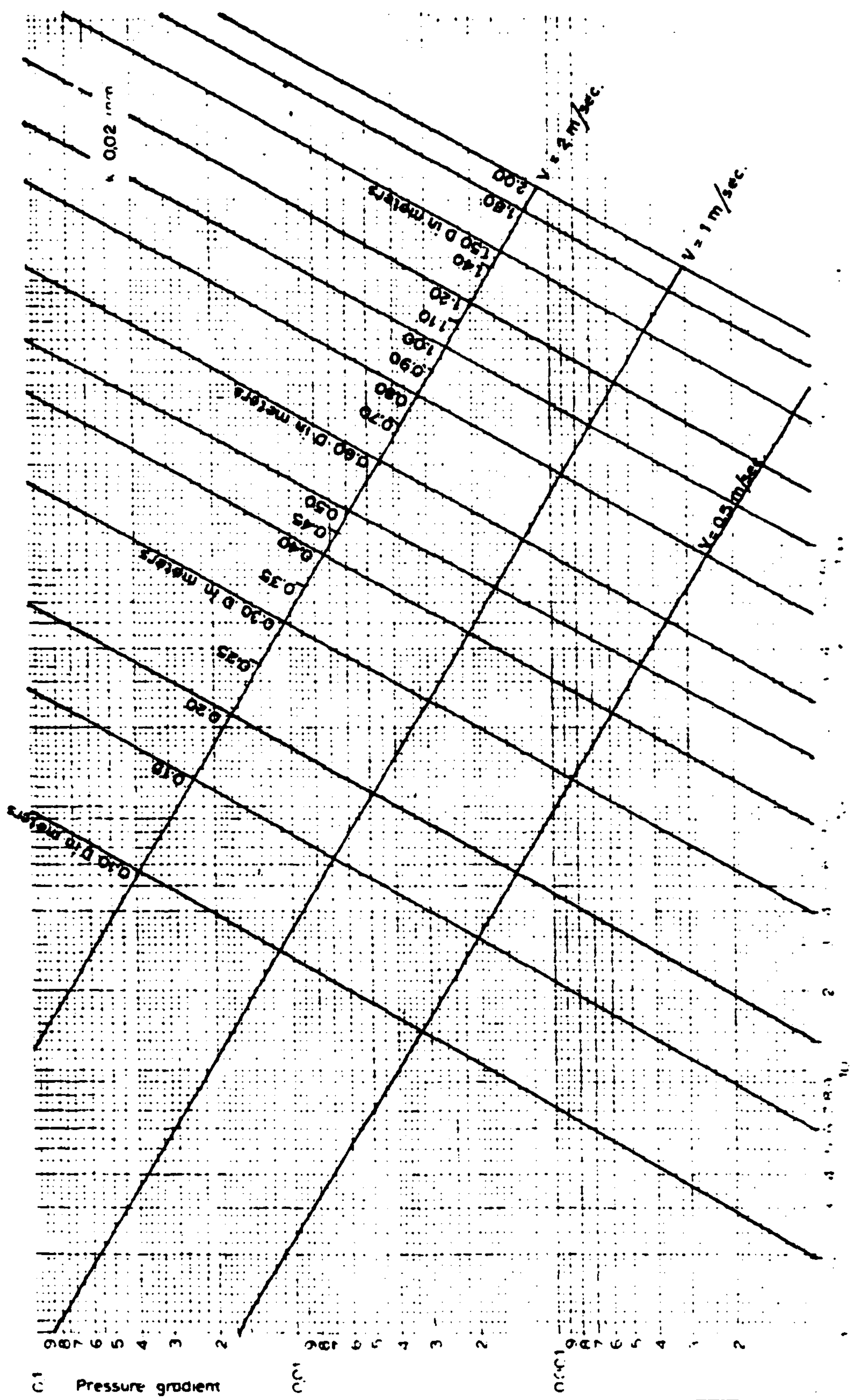

FIG 2b FLOW CHARACTERISTICS FOR PIPES
(temp 0°C wallroughness) k = 0,02 mm

WATER DISTRIBUTION

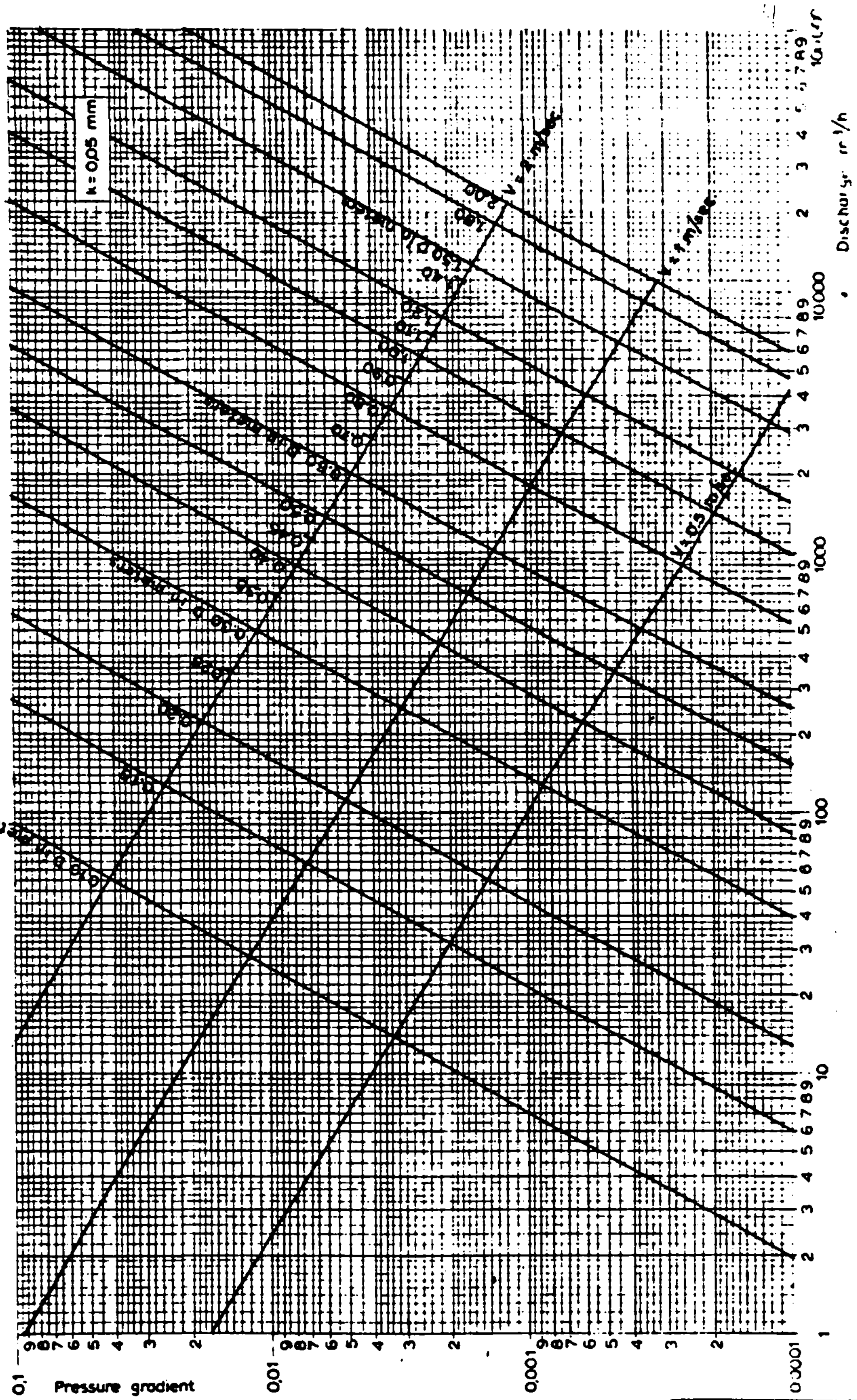

FIG. 2c FLOW CHARACTERISTICS FOR PIPES
(temp. 0°C wallroughness) k = 0,05 mm.

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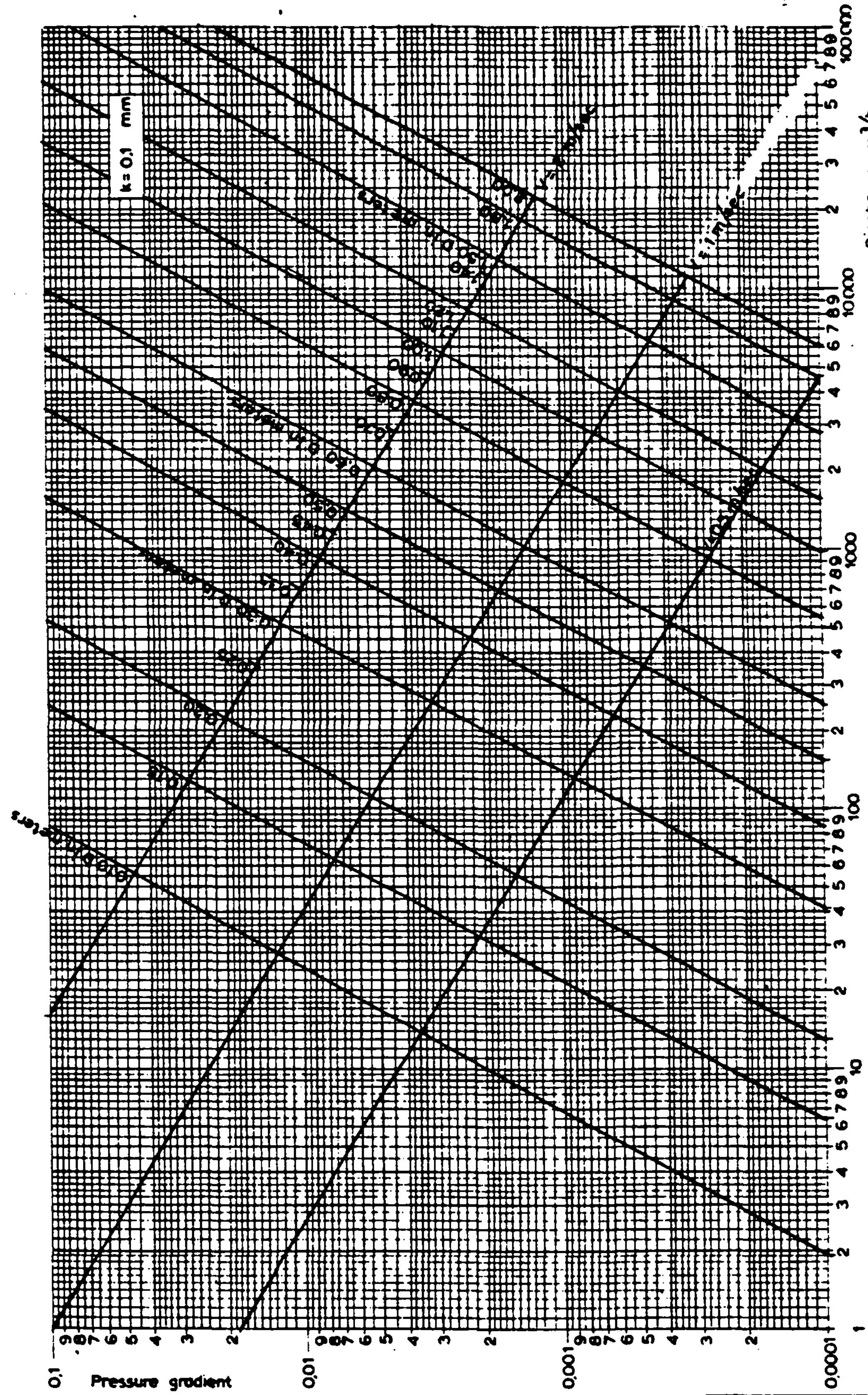

FIG. 2 d FLOW CHARACTERISTICS FOR PIPES
(temp. 0°C wallroughness) k = 0,1 mm.

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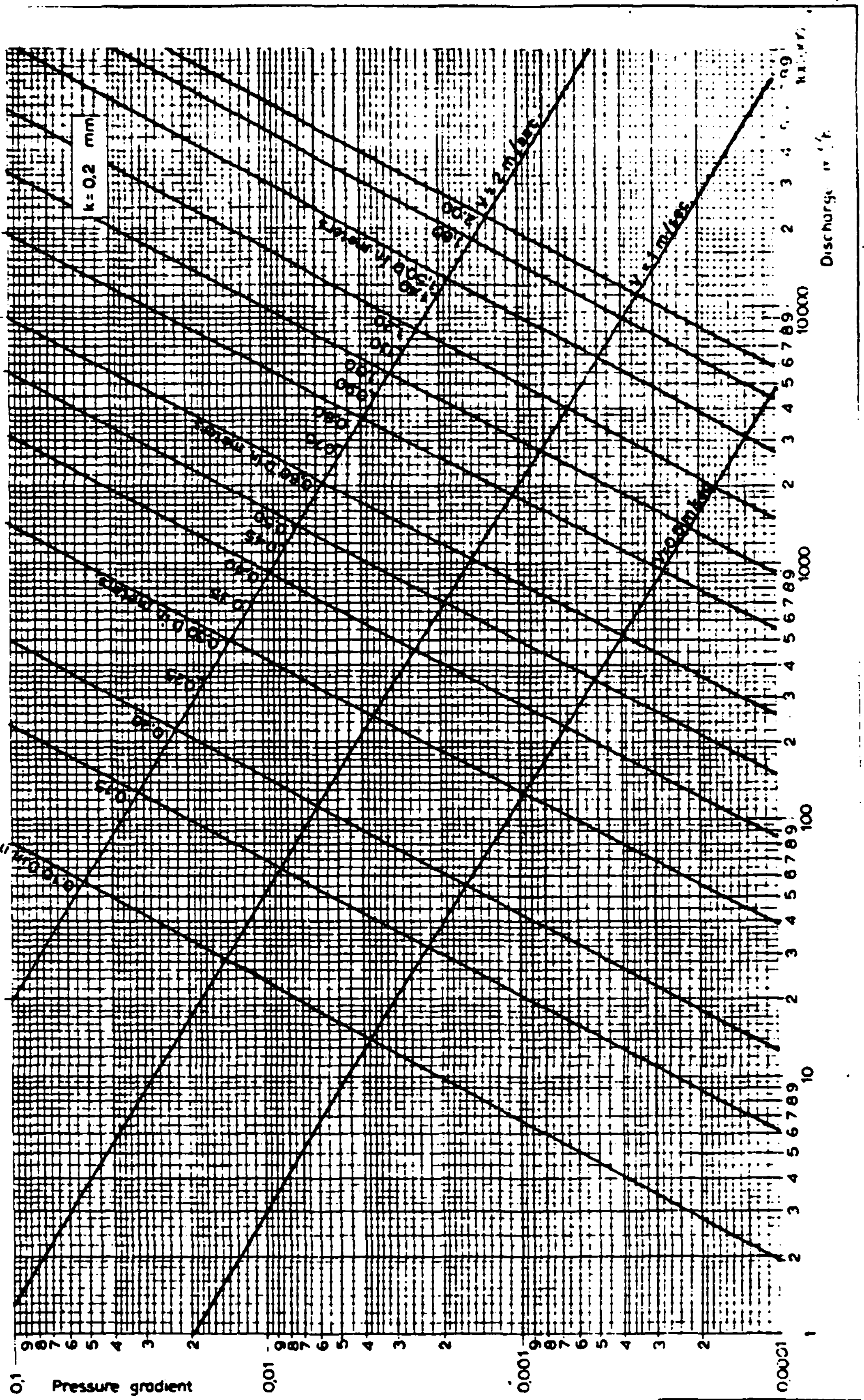

FIG. 2^{e} FLOW CHARACTERISTICS FOR PIPES
(temp. 0°C wallroughness) k = 0,2 mm.

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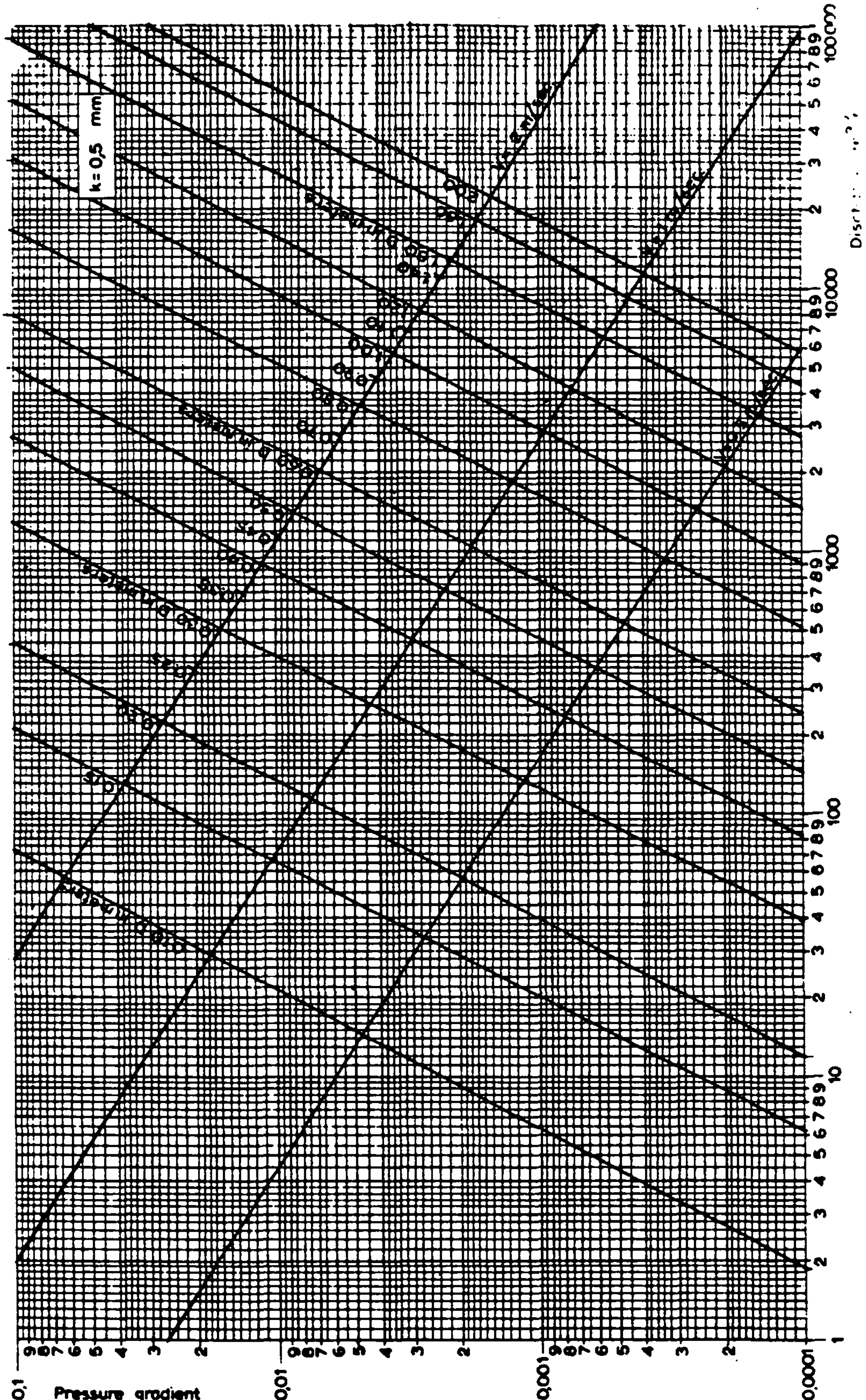

FIG. 2f FLOW CHARACTERISTICS FOR PIPES
(temp. 0°C wallroughness) k = 0,5 mm.

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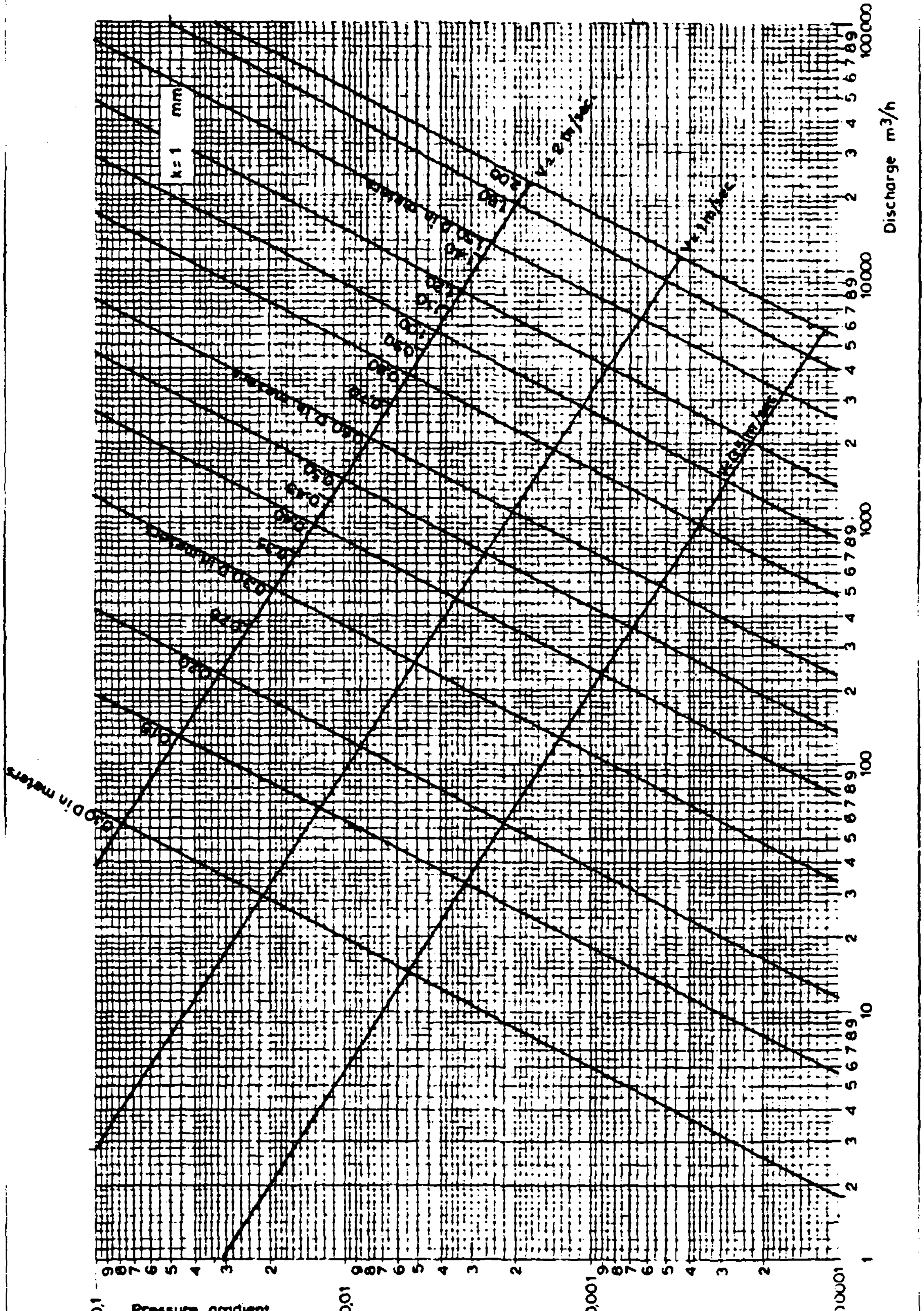

FIG. 28 FLOW CHARACTERISTICS FOR PIPES
(temp. 0°C wallroughness) k= 1 mm.

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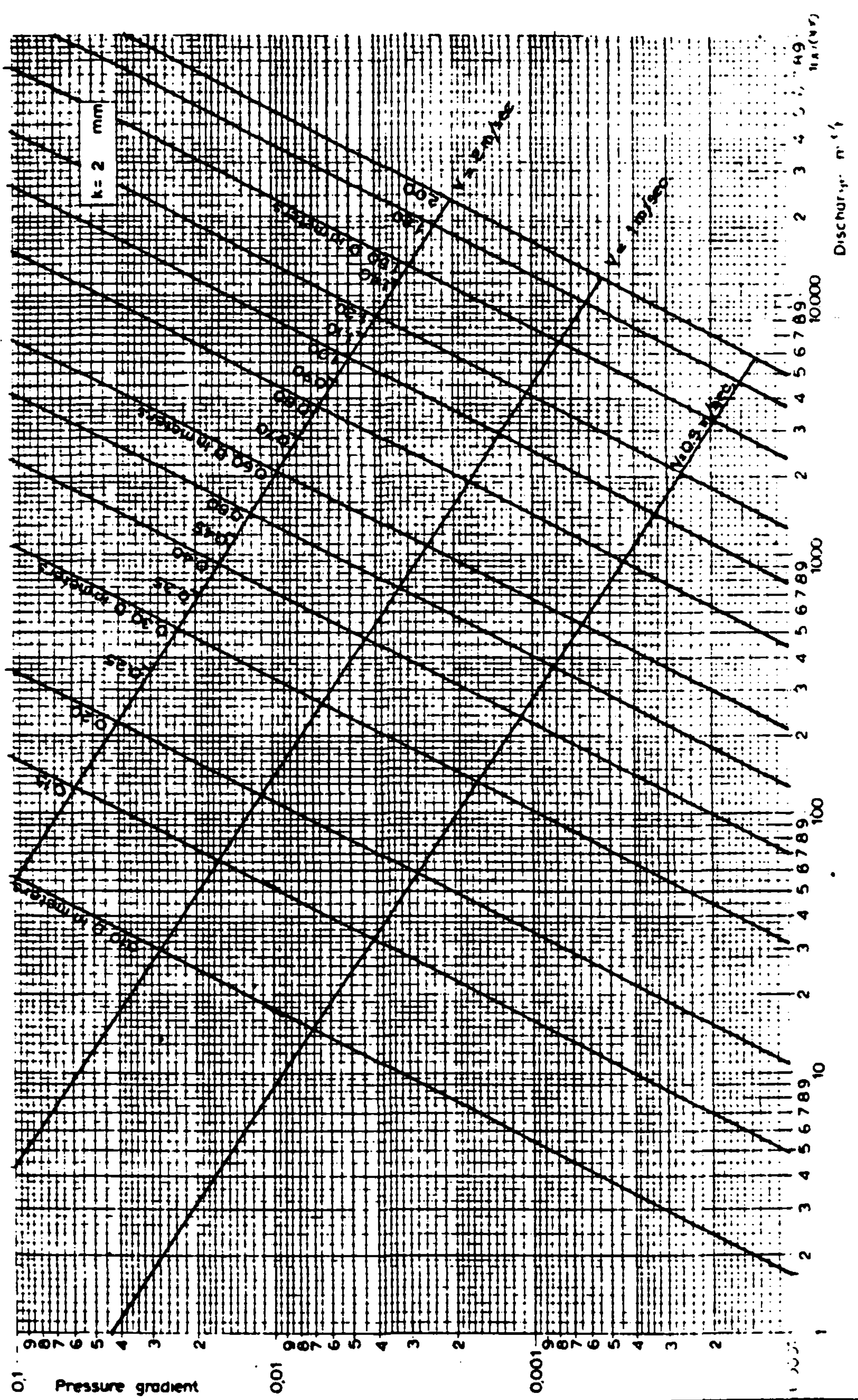

FIG. 2h FLOW CHARACTERISTICS FOR PIPES
(temp 0°C wallroughness) k= 2 mm.

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WATER DISTRIBUTION

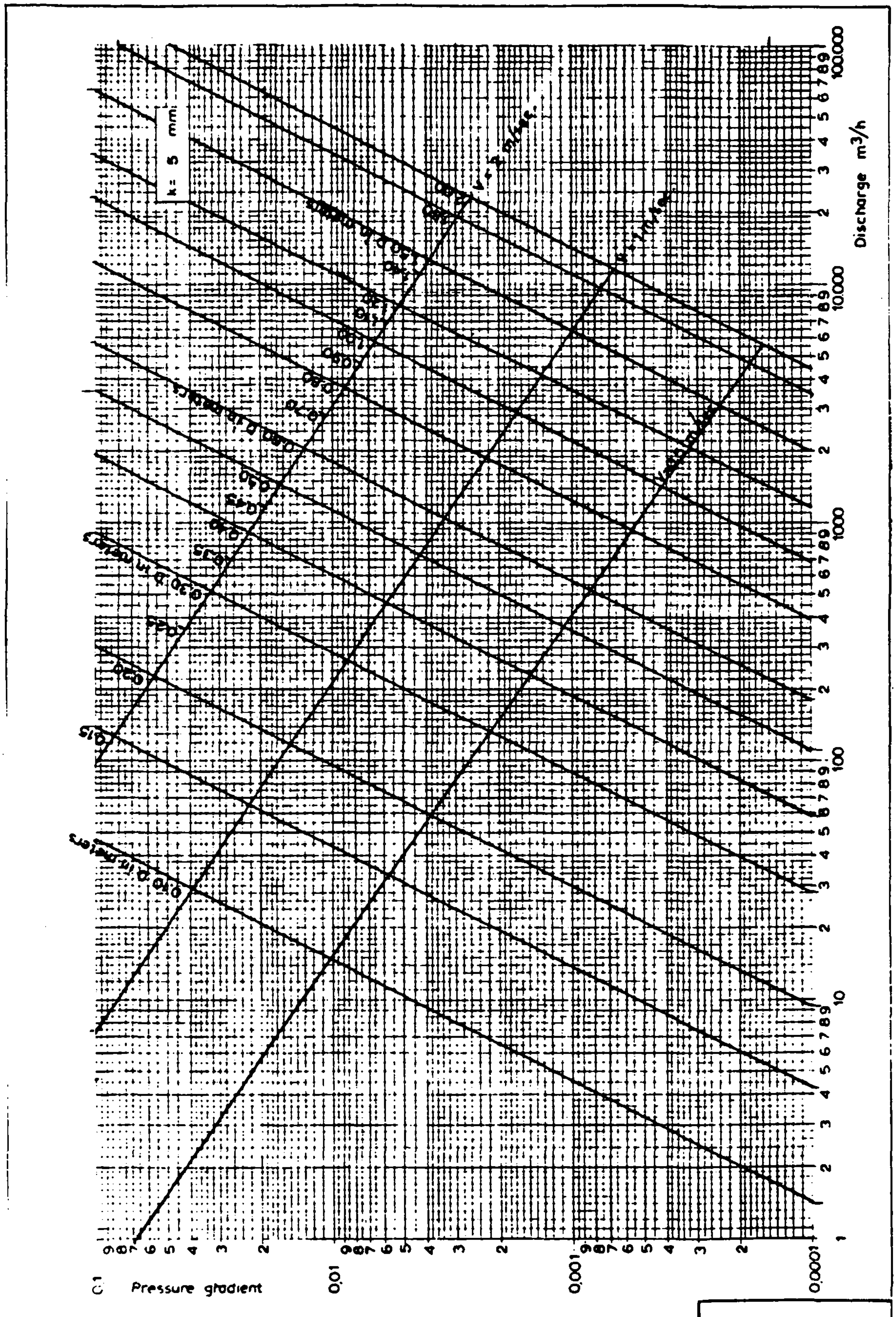


FIG 2 FLOW CHARACTERISTICS FOR PIPES
(temp ... roughness) k = 5 mm.

SANITARY COURSE
WATER DISTRIBUTION

QUALITY ASSESSMENT

Quality assessment is testing the product as to the quality requirements: the standard of the product. The question is, to what exent should there be a certainty that the delivered product will satisfy the relevant quality requirements.
In any manufacturing process there will be a deviation within certain limits, but they need not always to be unacceptable. Through the manner of quality assessment it can be determined to a considerable degree what certainty with regard to the absence of unacceptable deviations is necessary. But the cost of that certainty must also be considerable. In this area too consumers and producers will have a common interest as to the choice of the quality assessment system.
In the international context the above mentioned quality assessment systems are called certification systems.
Roughly - omitting a number of less relevant details - there are three basic systems:

1. Batch testing.
2. Type testing and assessment of factory quality control and its acceptance followed by surveillance.
3. Assesement of factory control and its acceptance only.

Batch testing is a system under which a batch of a product is sample tested and from which a verdict on the comformity with the specifications is isued.
The second system is based on a sample testing of the product according to a prescribed test method in order to verify the compliance of a model with a specification, with assessment and approval of the manufacturer's quality control arrangements,

followed by regular surveillance through inspection of factory quality control and audit testing of samples from both the open market and the factory.

The third system concerns one under which the manufacturer's capability to produce a product in accordance with the required specifications, including the manufacturing methods, quality control and type and routine facilities are assessed and approved, in respect of a discrete technology.

These three systems offer a variety of possibilities, and partly they have also certain limitations, advantages and disadvantages. It would lead too far to dwell on these. Yet each of them is indispensable in itself and they provide certain needs, depending on the nature of the product or the production method, the availability of generally accepted quality requirements and the nature of the application. However it might be, it is clear that for the development of any system the willingness to co-operate and mutal trust are primary preconditions. Whatever choice will be made, there will be a need for qualified personnel, measuring and testing equipment and an organization within which all that is going to function, in order to render the system operational. But not only that. One should also realise that the manufacturer is bound for a lot of development work within his company. The establishment of a quality assurance system - of which quality control, so necessary for certification, forms a part - will require investments. He would want to evaluate these burdens against the advantages such a system could yield to his company in the long term.

Certification is a means of improving industrial performance. It can be used as a tool to develop the quality of indigenously manufactured products as a means to substitue imports by giving the consumer a confidence in such products.
Especially for the manufacturer, a quality assurance system is a better way to improve industrial performance. Because only than it will be possible to subsitute an otherwise imperfect testing of the final product for a system enabling corrections and adjustmenst to be made at any moment during production.
Furthermore the results thus obtained may considerable contribute to the improvement of product design and of the production process itself.

Centralization of testing
After the conclusion that testing and inspection can be very helpful and recommendable it will be clear that the operation of testing and inspection by each water supply company itself is undesirable. The number, size and irregularity of testing and inspection activities make it uneconomical, not to mention the impossibility of permanent availability of testing equipment and laboratories.
The availability at the right moment and in the right place with the right expertise is almost impossible.
The continuity of testing and inspection is also important for the evaluation of the internal quality control of the manufacturer. A regular surveillance gives more than an instantaneous insight.

Centralization offers obviously the principal advantages for saving expenses, the availability of specialised expertise, more continuous information and the exchange of knowlegde and experience in the testing and inspection and in the application.

Summarized one can say that individual testing and inspection has remarkable restrictions which make it too expensive or organisationally and technically impossible. The limitations may be avoided in case a centralized system is chosen.

METHOD OF ASSESSING SOIL AGRESSIVITY

Method of assessing soil agressivity

Abbreviations		Factor
	I - Soil composition	
C	Chalky soil	+ 2
S	Sand	+ 2
CA	Chalky clay	+ 1
CM	Marly chalk	+ 1
SM or SA	Marly sand or clay	+ 1
LC	Silty chalk	+ 1
L	Silt	0
SAB	Sandy clay 75 % mud	0
SLB	Silty sand 75 % mud	0
LM	Silty Marl	— 1
LT	Peaty silt	— 1
A	Clay	— 2
AM	Marly Clay	— 2
H	Humus	— 2
M	Marl	— 3
TS	Thick silt	— 3
TB	Muddy ground	— 4
TM	Swampy ground	— 4
T	Peat	— 4
	II - Soil condition	
	Underground water present at level of pipe line	
ENP	— none	0
EP	— present	— 1
EV	— part time present	— 2
SN	Natural soil	0
Sr	Backfill	— 2
S = f	Same soil as trench	0
S ≠ f	Soil different to that of trench	— 3
	III - Measured specific resistance	
	> 10,000 ohms/cm	0
	10,000 to 5,000 ohms/cm	— 1
	5,000 to 2,300 ohms/cm	— 2
	2,300 to 1,000 ohms/cm	— 3
	< 1,000 ohms/cm	— 4
	IV - Water contents	
	< 20 %	0
	> 20 %	— 1

Abbreviations		Factor
	V - pH reading	
	pH > 6	0
	pH ≤ 6	— 2
	VI - Redox potential at pH 7	
	E > 400 mv	+ 2
	+ 200 to + 400 mv	0
	0 to + 200 mv	— 2
	E < 0	— 4
	VII - Concentration in carbonates	
	> 5 %	+ 2
	1 % to 5 %	+ 1
	< 1 %	0
	VIII - H2S and sulphides	
A	None	0
T	Trace	— 2
P	Present	— 4
	IX - Coal and coke	
A	None	0
P	Present	— 4
	X - Chlorides	
	Concentration:	
	< 100 mg/kg	0
	≥ 100 mg/kg	— 1
	XI - Sulfates	
	Concentration:	
	< 200 mg/kg	0
	200 to 250 mg/kg	— 1
	500 to 1000 mg/kg	— 2
	> 1000 mg/kg	— 3

Sum of factors	Aggressivity
> 0	Not aggressive
0 to — 4	Weakly aggressive
— 5 to — 10	Aggressive
< — 10	Very aggressive

THE USE OF THE EQUIVALENCE METHOD

Schematizing an area where a network has to be designed means that we are confronted with different categories of water consumption:
- low density areas i.g. with a relatively high water consumption per capita
- medium density areas
- high density areas i.g. with a relatively low waterconsumption per capita (public standposts!)

but also
- commercial areas
- industrial areas (heavy, i.g. much water consuming industrie, light industrie i.g. few water consuming)
- agricultural areas.

All different categories may have different consumption levels, water demand patterns and different peakfactors.
For the design hour these mentioned characteristics should be known or careful estimated.

In figure 1 examples of the mentioned consumption categories are presented with their schematized consumption pattern.

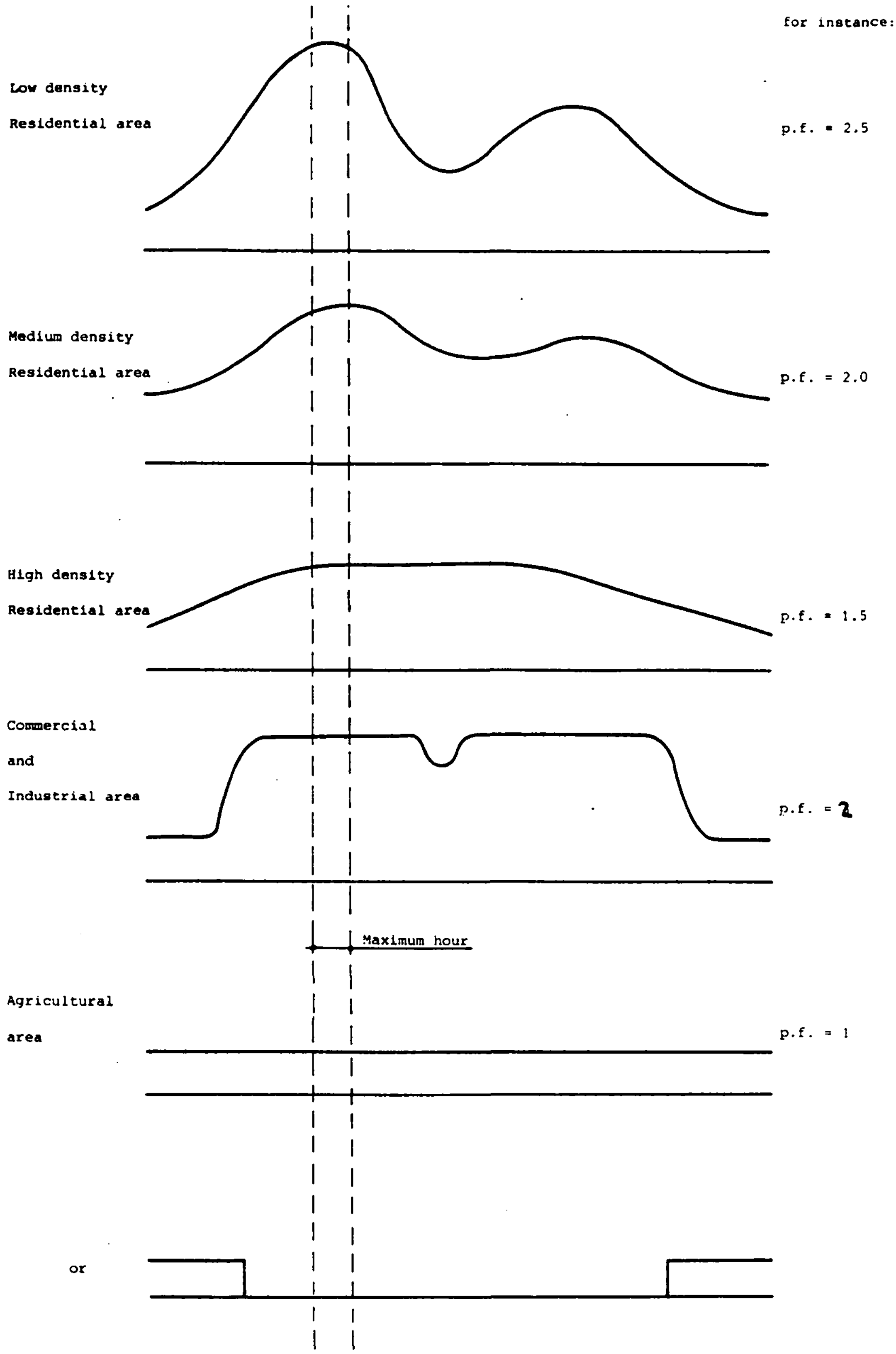


Fig. 1 - Different schematized water demand patterns for different consumption categories

From figure 1 may be concluded, if the schematized patterns are acceptable, that there is in general no big difference on which peak hour the design should be based.
Nevertheless there are different demand levels and peakfactors.

Designing means comparison of alternative variants. For the design of these variants the help of the equivalence method is most welcome because of the applicability.
It gives very quick a good insight in every considered variant and direct information about the required dimensions of the distribution mains.
The acceptable load of each main diameter is based on the predetermined permissable pressure gradients and is expressed in number of inhabitants.
In principle it is of course possible the use a range of tables of equivalence, all developed on base of the available pressure head (which also may be different for different areas) the considered water consumption level and the different peakfactors.

Nevertheless to express the influences of the different categories of water demand in a practical and acceptable way one has to try to limit the number of tools.

This limitation of number of tables can be reached by

a. making use of main loads, expressed in hectares (see table 1)

Areas	Population density c/ha	Per cap consumption l/c.d.	Peak factor	Per hectare consumption $m^3/h/ha$
low density, high income	50 - 100	200 - 250	2.4	1.0 - 2.5
medium density, medium income	100 - 150	150 - 200	2.2	1.4 - 2.7
high density, low income	150 - 200	50 - 150	2.0	0.6 - 2.5
very high density, very low income	300 - 500	20 - 50	1.8	0.5 - 2.0

Table 1 - Per hectare water consumption for different population groups

It may be concluded that the number of tables of equivalence indeed may be limited. For example one table of equivalence based as f.i. 2.5 $m^3/h/ha$ for each area is acceptable.
During the interpretation of the table it is good to keep the required accuracy in mind. Futural social changes, uncertain industrial establishment, policy changes regarding public standpost system transformed into house connections.
For the dimensioning of the distribution mains the considered areas are expressed in hectares.

b. Making use of population equivalents
It may be attractive to define a socalled "population equivalent unit" and express every con-

sidered area in this units.
Related to the situation in question the used population equivalent unit should be defined very well.
To be as flexible as possible the chosen unit (standard) preferably should be derived from a common category in the area, f.i. the medium density category.
For this category the next characteristics may be valid $q = 150$ l/c/d and the
peak factor $pf = 2.0$
The population equivalent unit will be then:

$$\frac{0.150 * 2}{24} = 0.0125 \text{ m}^3/\text{h}$$

In the high density area with f.i. $q = 100$ l/c/d and a peak factor of 1.5 this means that 1 capita requires $\frac{0.100}{24} * 1.5 = 0.00625$ m³/h or 0.5 population equivalent units.

In the low density area with f.i. $q = 250$ l/c/d and a peak factor of 2.4 1 capity requires $\frac{0.250}{24} * 2.4 = 0.0250$ m³/h or 2 population equivalent units.

In industrial areas may be calculated in an analoque way. F.i. the average daily water demand is 4000 l per laborer with a peak factor of 1.2.
One laborer in this way requires $\frac{4000}{24} * 1.2 =$ 0.200 m³/h or 16 population equivalent units.

For every area the actual peak consumption may be calculated which at the same time may be expressed in related number of population equivalent units. See table 2.

Area	Av. consumption	peak factor	peak consumption	Pop. equiv. units
residential area A	100	2	200	16000
residential area B	50	1.5	75	6000
industrial area C	150	1.2	180	14400

Table 2 - An example of some different areas with the related number of population equivalent units

With help of just one table of equivalence the suitable diameter of mains may be chosen now.
That table should be of course the table which was based on the chosen standard q = 150 l/c/d and a peak factor of 1.5.

A FURTHER INTRODUCTION TO THE HARDY-CROSS METHOD

Hydraulic background

First a recapitulation of the formulae is given.

In order to determine the head loss in a pipe caused by a flow of Q the formula of Darcy-Weisbach and the Colebrook formula is used.

$$\Delta H = \frac{\lambda L}{D} \frac{v^2}{2g} \quad \text{(Darcy-Weisbach)}$$

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left\{ \frac{k}{3.71 D} + \frac{2.51}{Re \sqrt{\lambda}} \right\} \quad \text{(Colebrook)}$$

For computation reasons instead of the formula of Darcy-Weisbach and of Colebrook the next modified formulae may be used:
for diameters < 400 mm

$$\Delta H = 0.1255 \; L \; D^{-5.25} k^{0.1695} \; Q^2 \qquad (1)$$

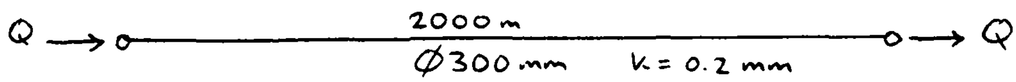
and for diameters ≥ 400 mm

$$\Delta H = 0.1241 \; L \; D^{-5.198} k^{0.1645} Q^2 \qquad (2)$$

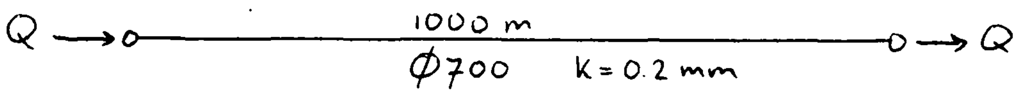
ΔH = head loss in [m]
L = length of the pipe in [km]
D = diameter in [m]
k = wall-roughness in [mm]
Q = flow rate in [10^3 m^3/h]

Both formulae are valid only under the following conditions:
0.3 m/s < velocity < 3.0 m/s
0.01 mm < wall-roughness < 2.0 mm

Some examples:



Q m³/hr	v m/sec	ΔH Colebrook m	ΔH formula(1)	Difference in %
40	0.155	0.19	0.17	12
60	0.240	0.40	0.38	4.7
80	0.310	0.70	0.68	2.9
100	0.390	1.08	1.06	1.9
200	0.790	4.40	4.25	0.9
400	1.57	16.00	17.00	- 5.9
1 000	3.93	92.00	106.20	- 13.4



Q m³/hr	v m/sec	ΔH Colebrook m	ΔH formula(2)	Difference in %
400	0.29	0.10	0.10	3
600	0.43	0.225	0.22	2.7
1 000	0.72	0.62	0.61	1.9
2 000	1.44	2.4	2.43	- 1.3
5 000	3.60	14.0	15.20	8
10 000	7.21	55.0	60.80	- 10

Hardy-Cross methods

There are two different iterative procedures:

1. the method of Balancing Flows.
2. the method of Balancing Heads.

Both methods will be discussed and illustrated with an example.

- Method of Balancing Flows

  In this method the continuity principle is considered in each grid of the network. (The network should therefore content only grids.)

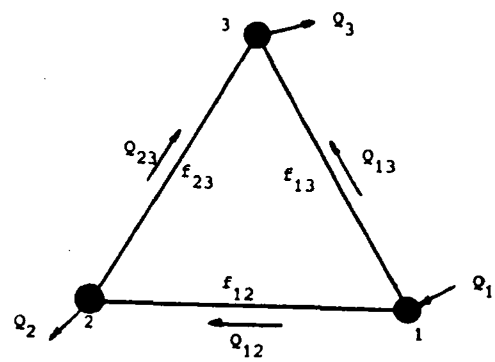


Fig. 1 - Balancing flow rates in a grid

The head loss in each pipe is: $\Delta H_{ij}=f_{ij}Q_{ij}^2$ (3)

Before a calculation is started of this network an assumption should be made for a positive flow direction, for example clockwise.

In the grid in figure 1 the algebraic sum of all three head losses should be equal to zero, resulting in the following condition:

$$\Delta H_{12} + \Delta H_{13} - \Delta H_{23} = 0 \qquad (4)$$

If the equation (3) is now used combined with equation (4)

$$f_{12}Q_{12}^2 + f_{13}Q_{13}^2 - f_{23}Q_{23}^2 = 0 \qquad (5)$$

Since it is known that Q_{12}, Q_{13} and Q_{23} are estimates of the real flow rates in the pipes, the left part of equation (5) is not equal to zero.

There is a certain deviation of zero and to obtain a better approximation of the true flow rate all the flow rates in the pipes must be corrected in the loop by ΔQ.

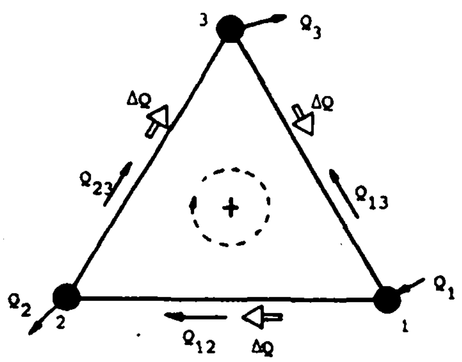


Fig. 2 - Correction flow rate in the considered loop

Consequently equation (5) becomes

$$f_{12}(Q_{12}+\Delta Q)^2+f_{13}(Q_{13}+\Delta Q)^2-f_{23}(Q_{23}-\Delta Q)^2 = 0 \qquad (6)$$

In this case it has been assumed that Q_{12} and Q_{13} have to increase and Q_{23} has to decrease.

$$f_{12}Q_{12}^2+2f_{12}Q_{12}\Delta Q+f_{12}\Delta Q^2+f_{13}Q_{13}^2+2f_{13}Q_{13}\Delta Q + f_{13}\Delta Q^2 - f_{23}Q_{23}^2 + 2f_{23}Q_{23}\Delta Q - f_{23}\Delta Q^2 = 0 \qquad (7)$$

Assume now that $\Delta Q^2 \sim 0$. Equation (7) then gives

$$f_{12}Q_{12}^2+f_{13}Q_{13}^2-f_{23}Q_{23}^2 = -\Delta Q(2f_{12}Q_{12}+2f_{13}Q_{13} + 2f_{23}Q_{23}) \tag{8}$$

$$\Delta Q = \frac{-(f_{12}Q_{12}^2 + f_{13}Q_{13}^2 - f_{23}Q_{23}^2)}{2(f_{12}Q_{12} + f_{13}Q_{13} - f_{23}Q_{23})} \tag{9}$$

The correction equation is:

$$\Delta Q = -\frac{\sum_{i=1}^{n} f_iQ_i|Q_i|}{2\,\Sigma\,|f_iQ_i|} \quad \text{or} \quad \Delta Q = \frac{-\Sigma\Delta H}{2\Sigma\left|\frac{\Delta H}{Q_i}\right|} \tag{10}$$

- <u>Example</u>

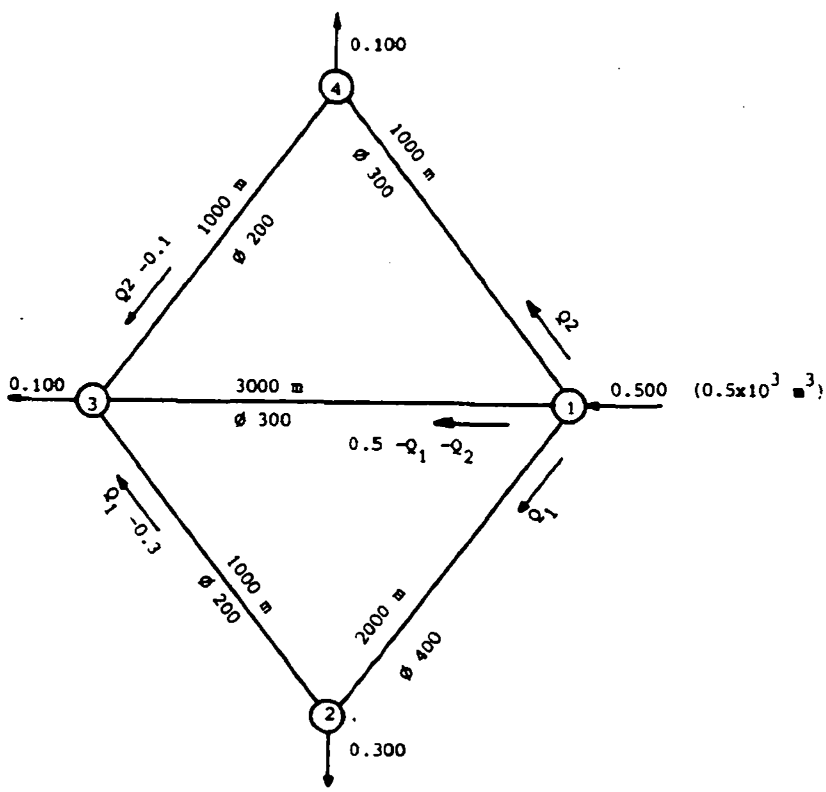


Fig. 3 - An example grid network to be calculated with help of the method of Balancing Flows

- Node 1 is a pumping station with a capacity of 0.5×10^3 m^3/hour.
- The water pressure in node 1 is 40 m above ground level.
- Levels of the nodes are:
  node 2: 10 m above ground level
  node 3: 5 m above ground level
  node 4: 10 m above ground level.
- The acceptable deviation in Q in each node should be < 0.0005 m^3/hour.
- Wall roughness k of the pipes is 0.2 mm.

The pressures and flows must now be calculated in the given network with the Hardy-Cross method of the Balancing Flows.

As can be seen in figure 3 there are two different grids:
grid 1 - 2 - 3
grid 1 - 3 - 4

Friction factors can be calculated with formula's (1) and (2)
f_{12} = 22 (2000 m of 400 mm)
f_{23} = 446 (1000 m of 200 mm)
f_{34} = 446 (1000 m of 200 mm)
f_{14} = 53 (1000 m of 300 mm)
f_{13} = 159 (3000 m of 300 mm).

With the calculation in the table the result is presented below:

	Grid 1 - 2 - 3			$Q_1 = 0.340$, $Q_2 = 0.140$, $\Delta_1 = +0.056$			$Q_1 = 0.284$, $Q_2 = 0.129$, $\Delta_3 = +0.0084$			$Q_1 = 0.2756$, $Q_2 = 0.1317$, $\Delta_5 = +0.0006$		
	Flow		Head loss	Q	ΔH	ΔH/Q	Q	ΔH	ΔH/Q	Q	ΔH	ΔH/Q
1-2	Q_1	+	$22\,Q_1^2$	0.340	+ 2.54	7.48	0.284	+ 1.77	6.25	0.2756	+ 1.67	6.06
2-3	$Q_1-0.3$	+	$446(Q_1-0.3)^2$	0.040	+ 0.71	17.75	- 0.016	- 0.114	7.14	- 0.0244	- 0.265	10.86
3-1	$-(0.5-Q_1-Q_2)$	-	$159(0.5-Q_1-Q_2)^2$	-0.020	- 0.06	3.18	- 0.087	- 1.20	13.83	- 0.0927	- 1.366	14.74
Σ 2 Σ					+ 3.19	28.41 56.82		+ 0.456	27.22 54.44		+ 0.038	31.65 63.31

	Grid 1 - 3 - 4			$Q_1 = 0.284$, $Q_2 = 0.140$, $\Delta_2 = -0.011$			$Q_1 = 0.2756$, $Q_2 = 0.129$, $\Delta_4 = +0.0027$			$Q_1 = 0.275$, $Q_2 = 0.1317$, $\Delta_6 = +0.0002$		
	Flow		Head loss	Q	ΔH	ΔH/Q	Q	ΔH	ΔH/Q	Q	ΔH	ΔH/Q
1-3	$0.5-Q_1-Q_2$	+	$159(0.5-Q_1-Q_2)^2$	+0.076	0.92	12.10	+0.0954	+ 1.45	15.17	0.0933	+ 1.384	14.83
3-4	$0.1-Q_2$	-	$446(0.1-Q_2)^2$	-0.040	- 0.71	17.75	-0.029	- 0.38	12.93	- 0.0317	- 0.448	14.13
4-1	$-Q_2$	-	$-53\,Q_2^2$	-0.140	- 1.03	7.36	-0.129	- 0.88	6.84	- 0.1317	- 0.919	6.97
Σ 2 Σ					- 0.82	37.20 74.40		+ 0.19	34.94 69.87		+ 0.017	35.93 71.88

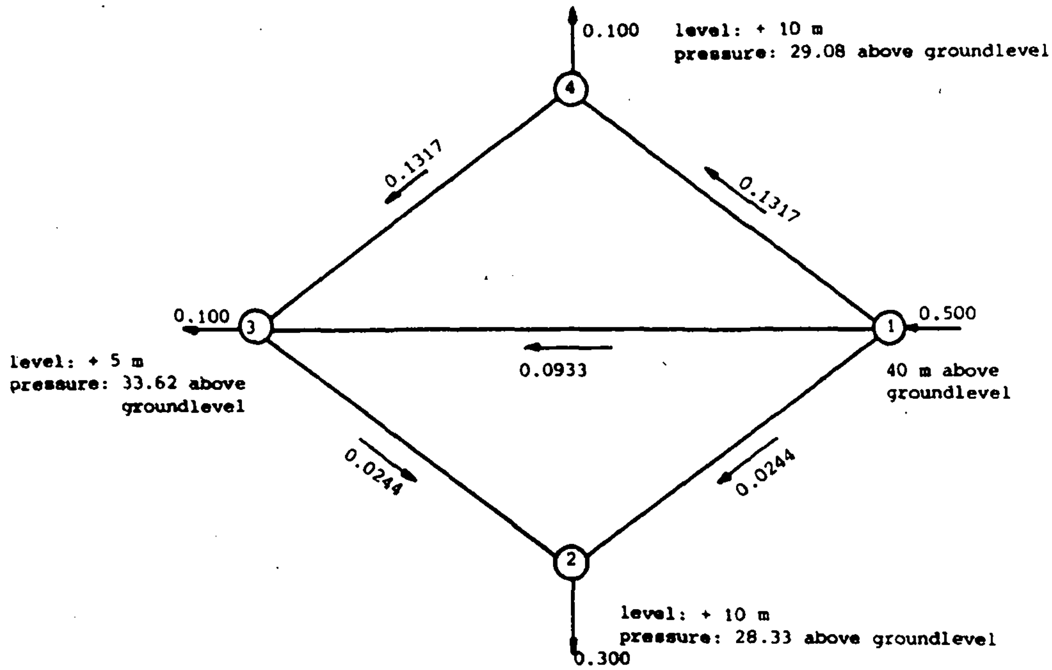


Fig. 4 - The result of the calculations

- Method of Balancing Heads

In this method the continuity principle is considered in each node of the network.

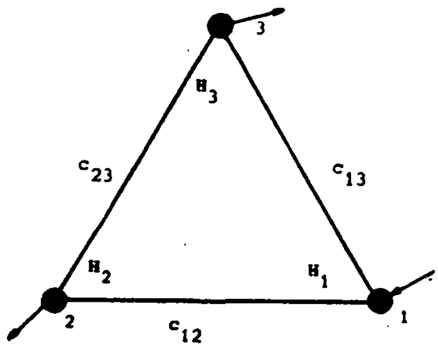


Fig. 5 - Balancing heads in the nodes of the considered grid

In each node there is the condition $\Sigma Q = 0$ (11)

The second equation used is $\Delta H = fQ^2$ (12)

In addition $Q = f^1 \sqrt{\Delta H}$ (13)

if $f^1 = \frac{1}{\sqrt{f}}$ (further $f^1 = c_{ij}$) (14)

The method of balancing flows uses estimates for the flowrates in the pipes of the network. In contrary with this method the method of balancing heads uses estimates for the pressure heads in the nodes of the network.

If we assume the pressure in node 1, H_1, in node 2, H_2 and node 3, H_3, then we can use the continuity equation in each node.

If the continuity principle is used in node 1 with the estimates of the pressure heads in nodes 1, 2 and 3:

$$Q - C_{12} \sqrt{|H_1 - H_2|} - C_{13} \sqrt{|H_1 - H_3|} = \Delta Q \qquad (15)$$

(Note that there must be the correct signs for the outgoing flow of the node).

Now change in- and outflow in such a way that $\Delta Q = 0$, by correcting $|H_1 - H_2|$ and $|H_1 - H_3|$ with a correction in node 1 of ΔH.

Then

$$Q - C_{12} \sqrt{|H_1 - H_2| - \Delta H} - C_{13} \sqrt{|H_1 - H_3| - \Delta H} = 0 \qquad (16)$$

$$Q - C_{12} \sqrt{H_{12} + \Delta H} - C_{13} \sqrt{H_{13} + \Delta H} = 0 \qquad (17)$$

The derivation may continue along several routes. An example of such a procedure will be elaborated here, considering just the first two terms of the binominal expansion of $(H \pm \Delta H)^{½}$ gives:

$$(H \pm \Delta H)^{½} = H^{½} \pm \frac{\Delta H}{2H^{½}} \pm \ldots. \qquad (18)$$

Using this approximation to remove the square root over ΔH gives:

$$Q - C_{12} \sqrt{H_{12}} - \frac{C_{12}\,\Delta H}{2\sqrt{H_{12}}} - C_{13} \sqrt{H_{13}} - \frac{C_{13}\,\Delta H}{2\sqrt{H_{13}}} = 0 \qquad (19)$$

$$\Delta H = \frac{Q - C_{12} \sqrt{H_1} - C_{13} \sqrt{H_{13}}}{\dfrac{C_{12}}{2\sqrt{H_{12}}} + \dfrac{C_{13}}{2\sqrt{H_{13}}}} \qquad (20)$$

$$\Delta H_i = \frac{\sum_{j=1}^{n} Q_{ij}}{½ \sum_{j=1}^{n} \{C_{ij}/\sqrt{H_{ij}}\}} \qquad (21)$$

- Example

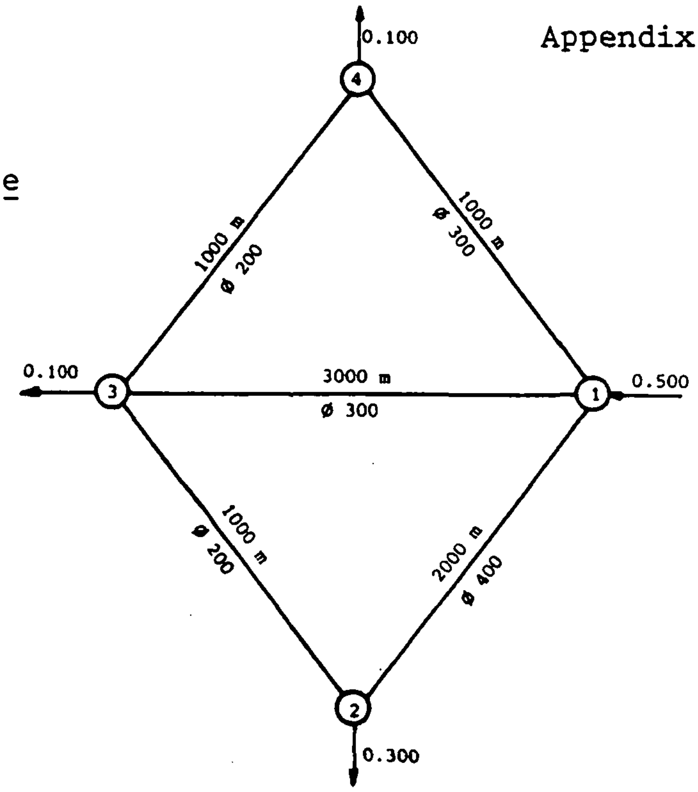


Fig. 6 - An example grid network to be calculated with
- the minimum allowable pressure is 20 m above ground level
- node 1 is a pumping station with a capacity of 0.5 10^3 m^3/hour
- the levels and estimates of head above ground level in each node are:

  node 2 10 m + ; 30 m +
  node 3 5 m + ; 29 m +
  node 4 10 m + ; 35 m +

Wall roughness k of the pipes is 0.2 mm.
The friction factors can be calculated with formulae (1) and (2) and $C_{ij} = \frac{1}{\sqrt{f_{ij}}}$

$C_{12} = 1/\sqrt{f_{12}} = 0.213$
$C_{13} = 1/\sqrt{f_{13}} = 0.079$
$C_{23} = 1/\sqrt{f_{23}} = 0.047$
$C_{14} = 1/\sqrt{f_{14}} = 0.137$
$C_{34} = 1/\sqrt{f_{34}} = 0.047$

Now calculate the pressures and flow rates in the network.

After 4 iterations the result is:

	Connection to node	C	H_i	ΔH	Q	$C/\sqrt{\Delta H}$	H_i	ΔH	Q	$C/\sqrt{\Delta H}$	H_i	ΔH	Q	$C/\sqrt{\Delta H}$
node 1	2	0.213	35.8	+ 4.2	- 0.436	0.0519	43.0	+ 2.24	- 0.318	0.071	43.63	+ 1.48	- 0.259	0.0875
	3	0.079	45.29	- 5.29	+ 0.182	0.0171	44.19	+ 1.05	- 0.081	0.0385	43.87	+ 1.24	- 0.087	0.0354
	4	0.137	44.41	- 4.41	+ 0.287	0.0326	44.40	+ 0.84	- 0.125	0.0747	44.22	+ 0.89	- 0.129	0.0726
	1		40.00	-			45.24		-	-	45.11	-	-	-
	Q_i				+ 0.500				+ 0.500		-	-	+ 0.500	
	Σ				+ 0.533	0.1016			- 0.024	0.1842			+ 0.025	0.1955
	Δ_1 = 5.24			H_1 = 45.24			Δ_1 = - 0.13		H_1 = 45.11		Δ_1 = + 0.127		H_1 = 45.23	
node 2	1	0.213	40	- 10	+ 0.673	0.0336	45.24	- 9.44	+ 0.654	0.0346	45.11	- 2.11	+ 0.309	0.0732
	3	0.047	29	+ 1	- 0.047	0.0235	45.29	- 9.49	+ 0.144	0.0076	44.19	- 1.19	+ 0.051	0.0214
	2	-	30	-	-		35.8	-	-		43.0	-	-	-
	Q_i	-	-	-	- 0.300	-			- 0.300				- 0.300	
	Σ				+ 0.326	0.0562			+ 0.498	0.0692			+ 0.06	0.0946
	Δ_2 = + 5.80			H_2 = 30 + 5.8 = 35.8			Δ_2 = 7.20		H_2 = 43.0		Δ_2 = + 0.634		H_2 = 43.63	

	Connection to node	C	H_i	ΔH	Q	$C/\sqrt{\Delta H}$	H_i	ΔH	Q	$C/\sqrt{\Delta H}$	H_i	ΔH	Q	$C/\sqrt{\Delta H}$
node 3	1	0.079	40	- 11	+ 0.262	0.0059	45.24	0.05	- 0.017	0.17	45.11	- 0.92	+ 0.075	0.0407
	2	0.047	35.8	- 6.8	+ 0.122	0.009	43.0	2.29	- 0.071	0.0155	43.63	+ 0.56	- 0.035	0.0312
	4	0.047	35.0	- 6	+ 0.115	0.009	44.41	0.88	- 0.044	0.0250	44.40	- 0.21	+ 0.021	0.050
	3	-	29.0	-			45.29		-		44.19		-	-
	Q_i				- 0.100	0.024			+ 0.100				- 0.100	
	Σ				+ 0.399				- 0.232				- 0.039	
	Δ_3 = + 6.29			H_3 = 45.29			Δ_3 = - 1.10		H_3 = 44.19		Δ_3 = - 0.32		H_3 = 43.87	
node 4	1	0.137	40.0	- 5	+ 0.307	0.0306	45.24	- 0.83	+ 0.124	0.074	45.11	- 0.71	+ 0.115	0.081
	3	0.047	45.29	- 10.29	+ 0.150	0.0073	44.19	+ 0.22	- 0.022	0.05	43.87	+ 0.53	- 0.034	0.032
	4	-	35.0	-	-		44.41	-	-		44.4	-	-	-
	Q_i	-	-	-	- 0.100	-			- 0.100				- 0.100	
	Σ				+ 0.357	0.0379			- 0.002				- 0.019	0.113
	Δ_4 = + 9.41			H_4 = 44.41			Δ_4 = - 0.016		H_4 = 44.40		Δ_4 = - 0.168		H_4 = 44.22	

Fig. 7 - The result of the calculations

After 4 iterations the result in m above ground level is presented in figure 8

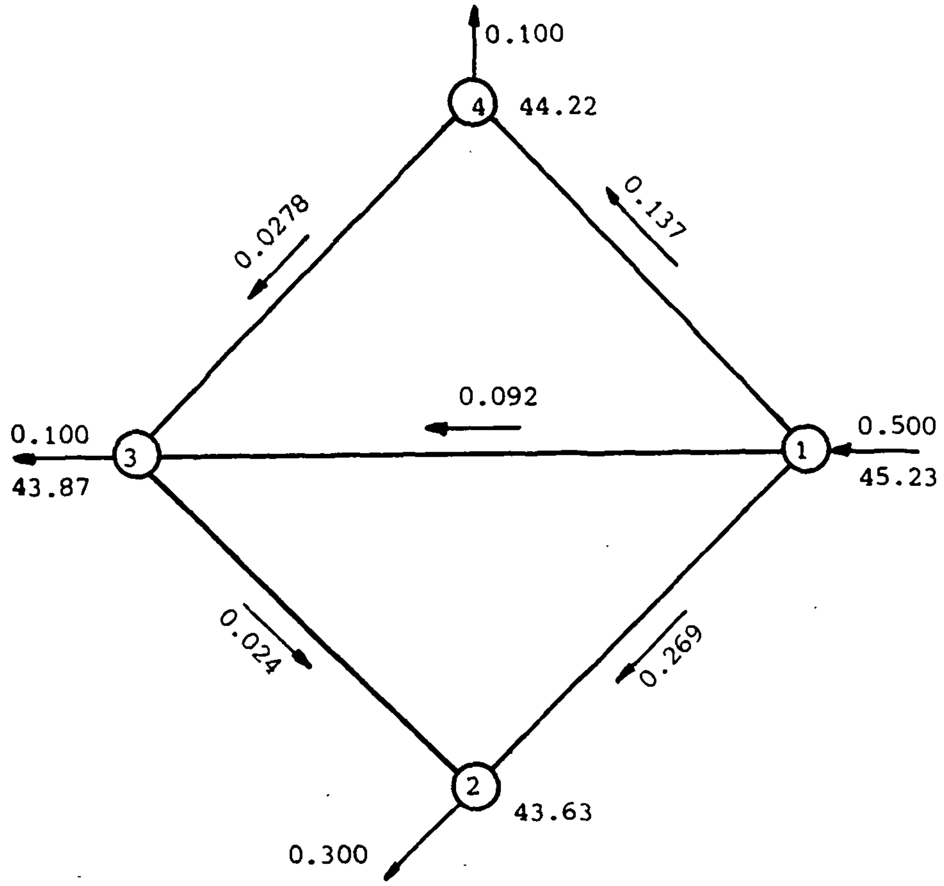


Figure 8 - The final results

If we consider the four continuity equations we conclude that there is a deviation left of 2-10 m^3 per node.

With the method of the balancing heads, the convergence at the end is a little slower than is the case in the first method.

In this second example the reason for this is mainly that the deviations in the first estimates were larger than the deviation of the flow estimates in the first example.