5.1 Introduction

Network calculation programs are used to simulate the behaviour of the network in reality. Figure 5.1 gives the complexity of the simulation process, which is an indication that results should be considered carefully. In this chapter a few applications of network calculations are demonstrated. Purpose is to show the process of modelling networks and demands and the possible pit falls in using network analysis and interpreting results.

Network calculations are applied in the process of (re)designing a network or to set up a strategy for operational management. Also in the day-to-day practise network calculations can be used. For instance to determine the effect of closing down a single pipe for repair. With computer simulation it is possible to calculate the behaviour of the network and analyse the effects of manipulations.
For all applications of network calculations the hydraulic relevance of the calculated results is dependant on the accuracy of the input data and the goal of the analysis. A pressurised drinking water network has pipes with a variety of internal diameters typically between 50 and 1500 mm.
Obviously the transportation capacity of the 50 mm is negligible compared to the transport capacity of the 1500mm pipe. Within a complete network subnetworks can be distinguished with specific hydraulic functions. The relative larger pipes form a network with the primary function to transport water over distances and the smaller pipes constitute a network with a basic function to distribute the water locally. In the interpretation of calculation results it is necessary to keep in mind the hydraulic relevance of a distribution network withinin the transport network.

Fig. 5.1 - Network model

Figure 5.1 gives an example of an all-pipe model of the network of the city of Amsterdam. No distinction is made in diameter. The pictures shows the complexity of a large network.
Characteristic for the design of networks is an uncertainty in boundary conditions induced by the difference between the expected lifetime of the pipes and the period that can be overseen to estimate the demand. The expected lifetime of a pipe, especially large ones, is decades. Meaning that the designed pipe will last much longer than the design period that can be overseen. It is very complicated to give a sound estimation of the demand situation within 10 to 15 years, which is only the beginning of the lifetime of the pipe. The tendency automatically is to design conservatively, using relative large safety margins.
Consequence is that the full capacity of transportation networks is only used rarely, basically only during maximum hour maximum day situations. Hydraulically the major effect will be that energy consumption drops because of less friction losses. For water quality reasons however an over dimensioned pipe is disadvanteous: low velocities and long residence times have a negative effect on water quality. These effects can be neutralised as much as possible with an adequate strategy for operational management. Network calculations are used to define such strategies, as will be demonstrated later on.

The hydraulic relevance of pipes in a network model will be calculated and two cases are shown dealing with both design and operation. The last demonstration is a detailed strategy for cleaning distribution networks.

5.2 Hydraulic relevance of pipes in a network model.

The formulas used in network calculation programs for pressurised networks are one-dimensional, meaning that only one variable can be calculated as demonstrated in chapter 4. The formula in its most simple form is the Darcy Weissbach formula in volume flow notation:

$$\Delta H = \frac{l\,L}{D}\frac{v^2}{2g} = 0{,}0826\frac{l\,L}{D^5}Q^2$$

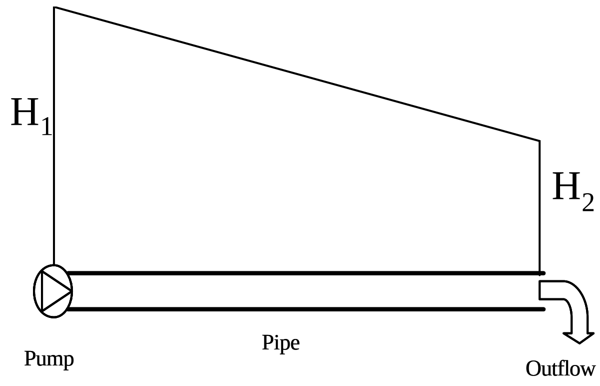

Fig. 5.2 - Schematic of calculation

The roughness of the pipe is translated to λ using the White-Colebrook equation (see chapter XX). Although dependent on the type of flow in the Reynolds number, this factor can be considered a constant pipe characteristic during the calculation.
Graphically the calculation is shown in figure 5.2 as one pipe with fixed characteristics, a pump at the one side and a demand on the other side of the pipe.

In a network pipes with different diameters are used. Dependent on the scale and purpose of the analysis the pipes can be distinguished on their hydraulic relevance for transport or distribution. An example is used to demonstrate this.

Assume a network of two parallel pipes, joint together at the beginning and the end of the pipes. Length and roughness are the same, the diameter of the first pipe is D and the second pipe xD, with x a factor larger than zero (fig. 5.3).

Pressure drop over both pipes is equal according to Kirchhoff's law (pressure drop over a loop is zero) resulting in:

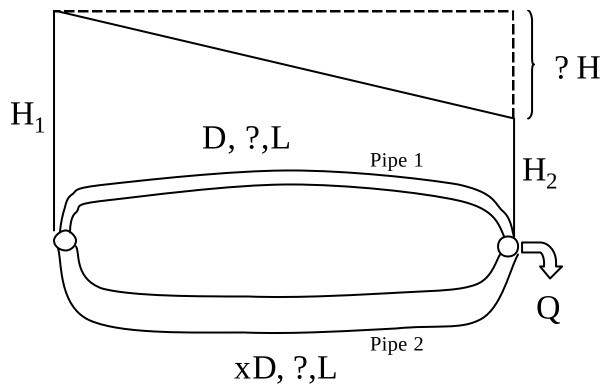

Fig. 5.3 - Two pipes with relevant data (l , L, D and Q) x > 1

$$\Delta H_1 = \frac{l\,L}{D^5}Q_1^2 = \Delta H_2 = \frac{l\,L}{(xD)^5}Q_2^2 = \Delta H$$

With some manipulation the rate between the volume flows can be related to the rate of the diameters x:

$$\frac{\cancel{lL}}{\cancel{D}^5}Q_1^2 = \frac{\cancel{lL}}{(x\cancel{D})^5}Q_2^2 \Rightarrow \frac{Q_2}{Q_1} = x^{2,5}$$

The rate between the volume flows is to the power 2,5 dependant from the rate between the diameters of the pipe. Graphically this is shown in figure 5.4.

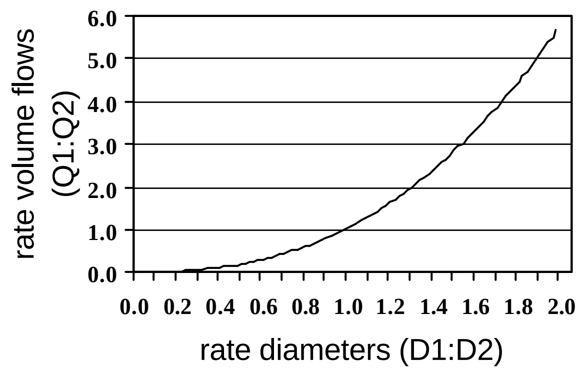

Figure 5.4 - rate diameter versus rate volume flow

Dependant on the desired accuracy of the hydraulic calculation a range of relevant diameters can be derived. If this is 10% than the smallest diameter is 0,4 times the largest. With a factor 2 between the diameters the volume flow varies with a factor 5,6.

The goal of the calculation should be well defined to select the relevant range of data. With modern database technology it is often possible to create all pipes models to make calculations. A large amount of data however can 'suffocate' the calculation, because only a relative small amount of calculated results are relevant for the goal of the calculation. Moreover an abundance of data gives an untrue feeling of accuracy.

5.3 Design of main transport network

The case study used is the design of a major transport line, which will carry over 11 million cubic meters of water a year. In the north of the network a production plant will be closed, because of water quality reasons. The replacing capacity will be situated in the south of the system, so the water should be transported over a distance of approximately 30 kilometres. Fig. 5.5 gives a representation of the transportation network.

Taking into account a seasonal and daily fluctuation, the maximum hourly transport capacity will be around 3400 m³/h.: The average flow will be:

$$\frac{11.000.000}{365*24} = 1256\ \mathrm{m}^3/\mathrm{h}$$ and with a maximum hour factor of 1,8 and a maximum day factor of 1,5 this gives a maximum hour flow of 3400 m³/h.

For a first estimation of the desired diameter a pressure slope of 1 mWc/km is assumed (A pressure drop of 1 Meter Water Column (100 Pascal) per kilometre of pipe or 0,001 m/m, see text box). The λ is estimated as 0,02.

$$\frac{\Delta \mathrm{H}}{\mathrm{L}} = 0,0826\frac{l}{\mathrm{D}^5}Q^2 = 0,001$$

$$= 0,0826\frac{0,02}{\mathrm{D}^5}\left(\frac{3400}{60*60}\right)^2 \Rightarrow \mathrm{D} \approx 1,08\ m$$

In a calculation for this problem the pipes with diameters starting from 300 mm. will be hydraulically relevant. This makes it possible to make a model with a limited amount of pipes. For the actual calculation time this doesn't make so much difference on modern computers, but for the interpretation it will. Using an all pipe model will complicate the handling of data,

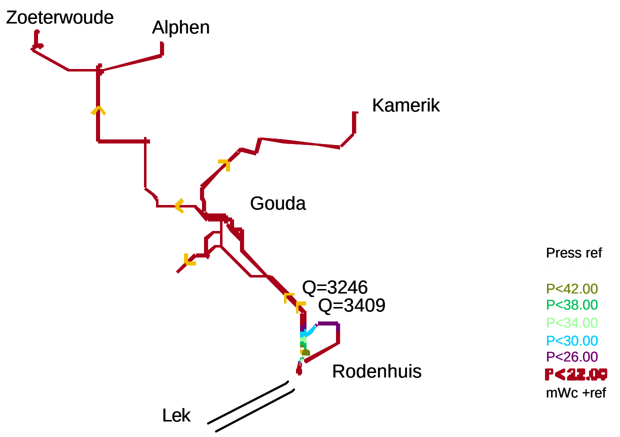

Fig. 5.5 - Picture of situation no extension

Rules of thumb for design drinking water pipe lines

For the first rough design of drinking water pipe lines simple rules of thumb can be applied to have a first impression of the lay out of the network.
For transport lines over larger distances the total pressure drop is the most important variable. The slope of the pressure drop is the most commonly used parameter in the order of 1 to 2 meter per kilometre pipe (1m/km or 0,001 m/m). This means that with one typical pump stage of 50 tot 60 mWc a distance of 20 to 30 kilometre can be overcome. Larger pressure drops over these distances requires a booster pumping station.
For smaller pipes over smaller distances a higher slope can be allowed, for instance 2 tot 5 m/km.

Also the velocity can be used as a parameter for a first dimensioning. For almost all lines a maximum velocity of 1 m/s is a good first estimation. Velocities should not drop too far, because the adverse effects on water quality. Low velocities will allow for more sedimentation than is desirable.

Pressure in a network in the Netherlands will be between a maximum of 60 mWc and a minimum of 20 to 25 mWc. In more hilly areas the maximum pressure can be much higher as a result of the difference in ground level. If pressure comes above 60 mWc regularly one should consider pressure release equipment. First to reduce leakage losses: a high pressure will cause more leakage than a lower pressure. Secondly to give more comfort to customers. High pressures make it difficult to dos on taps and showers. Moreover the danger for local water hammer will increase with all the adverse effects.

introducing errors and a lot of non-relevant information.
To skeletonise a network, suppliers of network calculation software offer automated skeletonizers. The use of these skeletonizers is explained in paragraph 5.8.

A hydraulic model of the system with the relevant pipes is made up and given in figure 5.5.
The first calculation made is the analysis of the system without any extension. This is to assess the ex-

tra capacity of the system. In figure 5.5 the pressure situation is shown for the maximum hour of the maximum day. As lowest allowable pressure in the system 200 kPa is assumed (20 mWc). Pressures below this are coloured red.

With regard to the extra amount of water to be transported (3400 m3/h) one can see that it is more than doubled and pressure very quickly drops below the threshold of 20 mWc.
So as could be expected the pressure situation is not adequate. There is a need for a new transport line. Three alternatives are considered:

- o An 800 mm. pipe with a booster pumping station half way. This allows for a larger pressure slope than 1 m/km (figure 5.6), but is compensated with an extra booster.
- o A 1000 mm. pipe without any extra pumping (figure 5.7).
- o A 900 mm pipe with a smaller booster pumping station half way. (figure 5.8).

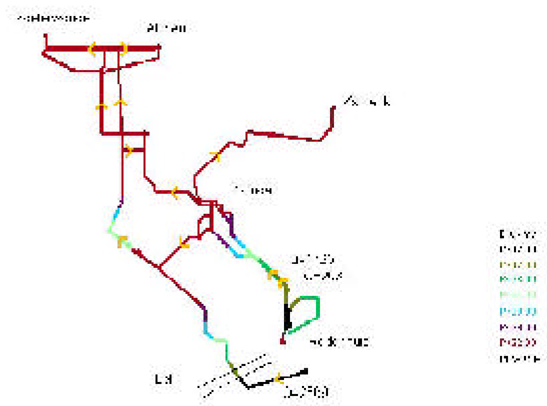

Fig. 5.6 - Alternative 1 (800 with booster)

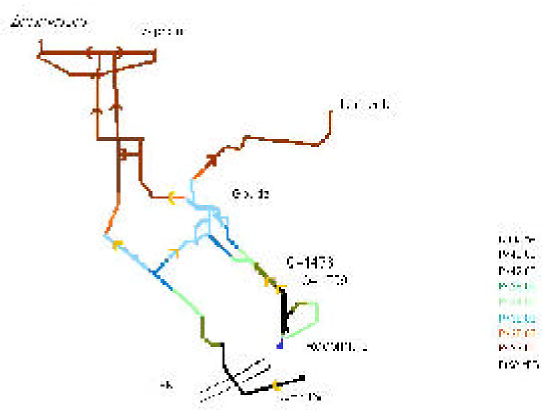

Fig. 5.7 - Alternative 2 (1000 without booster)

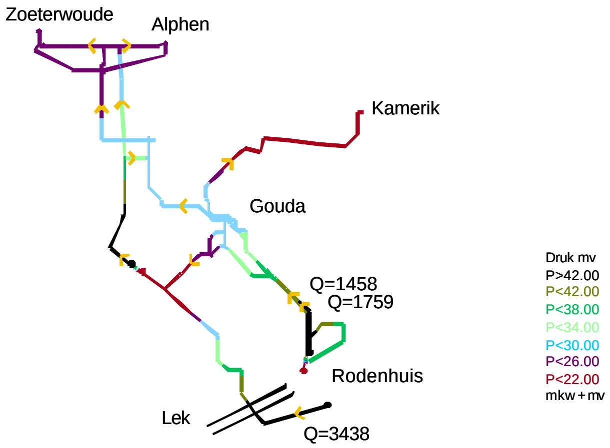

Fig. 5.8 - Alternative 3 (900 with booster)

In the first alternative the pressures are just not high enough. (The flow directions are marked with small arrow heads). The booster in the pipe is recognisable because the pressure increases in the flow direction.
Increasing the pressure of the booster is not a good option. The pressure before the booster is already too low. Also the transport capacity of the pipe is too small. A significant volume flow is transported through the original network causing low pressure.
The second alternative is to minimise the pressure drop over the new pipe.
In the second alternative a larger pipe (1000 mm) is implemented without any extra boosting (the pressure drops over the pipe line in streaming direction). Larger parts of the network now have a sufficient pressure, but still the extremities have insufficient pressure. As they are all above the first major city (Gouda) boosting with the help of a booster pumping station is the best way. Moreover the size of the pipe is declined slightly.
The third alternative is a 900 mm pipe with a booster station.
The entire network now is under sufficient pressure using a booster in the middle of the new line. (The smaller pipe in the north east of the network will be pressurised by an extra small booster. This was however not a part of the original problem, but something that came up in the analysis when future demands were projected in the network)
The final choice for the third alternative is not a technical choice alone. Also Alternative 2 could suffice, if a small booster station was implemented. The reason for choosing the smaller 900 mm. pipe and booster station is on the one hand a financial one: the price of a pipe is partly dependent on the diameter of the pipe. The second reason was the fact that

for the larger pipe a licence procedure had to be started, that could take a long time, including an environment impact analysis. For the smaller pipe this analysis was not necessary.

5.4 Strategy for operational management: residence time analysis

A strategy for operational management of a network is aimed at conserving water quality as best as possible. Two key hydraulic factors influencing water quality are residence time and velocity. Residence time itself is not the parameter that determines water quality deterioration, but the biological and chemical processes in the network are mostly time dependant. The longer the residence time in a network, the greater the affection of the water quality will be. Velocity influences water quality because at low velocities fine sediment will settle at the bottom of the pipe and possible whirl up again at higher velocity rates.

Over dimensioning of trunk mains is not uncommon, but is the main reason for low velocities and to a lesser extent for long residence times. An important reason is the long lifetime of a pipe, much longer than the design period that can be overseen in the design phase. In the period that the demand is 'growing' towards the designed capacity the flow through the pipe is low and will cause long residence times with the mentioned negative impact on water quality.

For reliability reasons transportation systems are often designed as looped systems to assure supply even during shut off of part of the system (see also Reliability chapter 6). A disadvantage of looping is that water can commute in the periphery of the system. Theoretically one drop of water will stay in the network forever in a looped system.

In the following case a transport network is analysed during the maximum supply situation and during the minimum supply situation. Goal is to analyse where low velocities and long residence times exist.

The largest existing pipes in this case determine the hydraulic relevance of the pipes, which in this case was 500 mm. This resulted in relevant pipes with diameters starting with 200 mm. The demands are relevant on the same level, so demand nodes are formed, summarising the demands and putting them on the nodes.

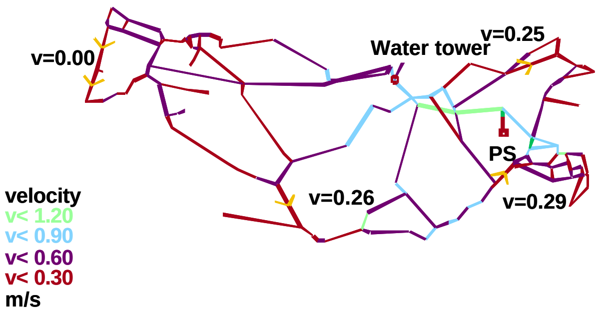

Fig. 5.9 - Flow relocation in maximum situation

The network is fed at two points. One is a pumping station (PS) the other is a fixed pressure point in the form of a water tower. This means that the tower is dependent of the pressure situation either a feeding point or a demand point.

This network is a typical long distance semi-rural network, because of the long distances and the according low velocities. Fig. 5.9 give the calculation results for the maximum demand situation. The colors represent the flow velocities. The friction losses in this network are minimal and so the velocities are low as well. The highest velocity is found in the south-east part and is 0,29 m/s. Also the flow directions are indicated with the help of small arrows. Not the very low velocity in the western part of the network.

In the minimum situation (fig. 5.10) the velocities are almost in the entire network coming down to less than 0,1 m/s. At all the indicated areas the flow direction reverts between the maximum and the minimum situation. The analysis results in the identification of areas that have low velocities during maximum hours. During the minimum hours the flow direction reverse and is even lower and actually pointing at commuting water. This indicates that in the low flow zones actual 'dead' water may occur during the reversal of flow. The residence time of the water

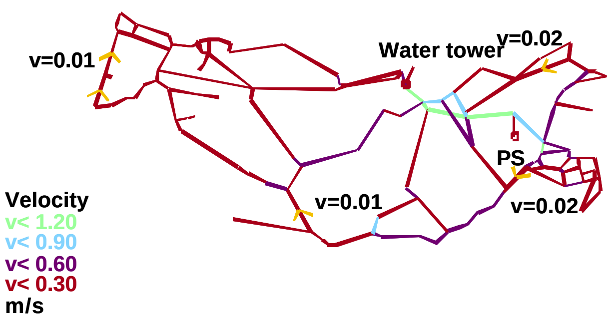

Fig. 5.10 - Flow relocation in minimum situation

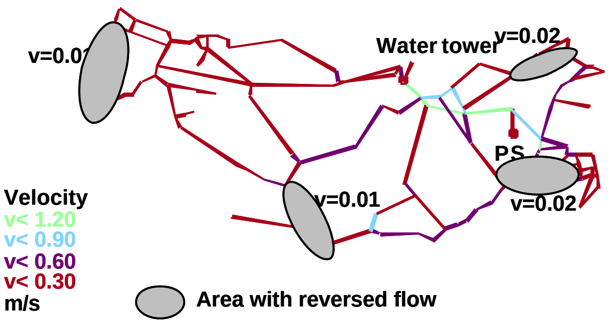

Fig. 5.11 - Zero flow zones

can get very long as result of this commuting.

In figure 5.11 the areas that will be affected because of the low velocities and commuting water are indicated.

Remedies for this situation depend on the severity of the problem. The starting point is the water quality leaving the treatment plant. Is this chemical and biological stable water, than residence time can be longer than with more unstable water. If there is a need to actually take measures, the possibilities are to shorten the residence times or to clean the network regularly.
Shortening of residence times can be accomplished by shutting of some parts of the looped network to avoid the commuting water. Care must be taken to avoid pipes that have no flow what so ever. The part between the last consumer and the closed valve will not be refreshed at all.
Another measure can be the cleaning of the network, to remove the sediments that accumulate during the low velocity periods and influence the biological activity. This will be demonstrated in the next paragraph.

5.5 Strategy for operational maintenance: cleaning of networks

An application of network calculations on a detailed level is the calculation of cleaning programs for distribution systems.
The most common technique used in the Netherlands is flushing with water. The desired mechanism is the increase in flow velocity, whirling up loose sediments and removing them with the water flows. Effective cleaning or flushing of a pipe meets the following boundary conditions:

- Velocity at least 1,5 m/s
- Water originates from a clear water pipe
- Flushed volume is at least 2 times the contact of the pipe.

Meeting these requirements is very important. Designing a flushing program starts with the identification of the clear-water-front and the cleaning progresses from this point, dragging the clear-water-front into the network.

To design a flushing program an all pipe model is necessary, as far as the 'hard' data as length diameter and roughness are concerned. The hydraulic relevance of the 'soft' data as on demand are less important. The volume flows induced with flushing are so much larger than normal flows induced by average demand, that they can be ignored.

In Annex 5.1 a descriptions of a special tool called the Flush Planner is demonstrated.

5.6 Storage facilities in a network

5.6.1. Volume of storage

The main purpose of storage facilities in the transport and distribution is to balance the constant production of a treatment plant and the varying demand in a network. For water quality and capacity reasons a treatment plant should produce at a constant level. Over a certain period, mostly 24 hours, the production level is constant at the average demand that is forecasted for this next 24 hours. During the night hours, when demand is lower than the average demand the surplus is stored in reservoirs. During hours of above average demand, the stored volume is used to replenish the deficit. Figure 5.12 shows a typical demand curve, the average demand and the stored

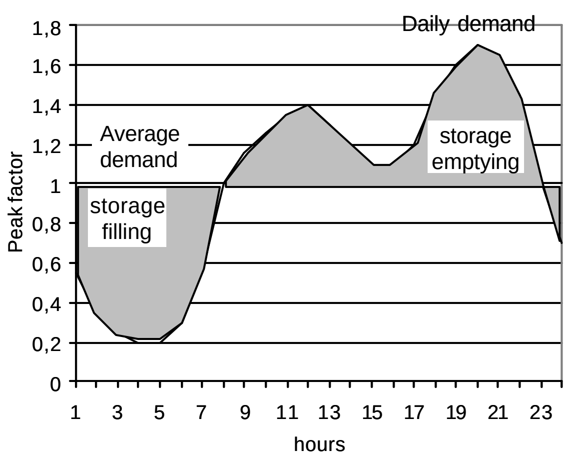

Fig. 5.12 - Storage volume

Table 5.1 - Peak actors and storage volume

Hour of the day	Demand factor	Average peak factor	Average – demand (volume flow to storage)	Volume storage
1	0,55	1,00	0,45	0,45
2	0,35	1,00	0,65	1,10
3	0,25	1,00	0,75	1,85
4	0,20	1,00	0,80	2,65
5	0,20	1,00	0,80	3,45
6	0,30	1,00	0,70	4,15
7	0,55	1,00	0,45	4,60
8	1,00	1,00	0,00	4,60
9	1,15	1,00	-0,15	4,45
10	1,25	1,00	-0,25	4,20
11	1,35	1,00	-0,35	3,85
12	1,40	1,00	-0,40	3,45
13	1,30	1,00	-0,30	3,15
14	1,20	1,00	-0,20	2,95
15	1,10	1,00	-0,10	2,85
16	1,10	1,00	-0,10	2,75
17	1,20	1,00	-0,20	2,55
18	1,45	1,00	-0,45	2,10
19	1,60	1,00	-0,60	1,50
20	1,70	1,00	-0,70	0,80
21	1,65	1,00	-0,65	0,15
22	1,45	1,00	-0,45	-0,30
23	1,00	1,00	0,00	-0,30
24	0,70	1,00	0,30	0,00

and replenished volume.

In table form this is represented in table 5.1

The difference between the average flow and the actual demand is the flow to or from the storage. A positive flow is going into the storage and a negative flow is coming out of the storage. The cumulative flows give the needed volume of the storage. The largest needed volume is +4,6 times the average hourly flow. The smallest needed volume is –0,3 times the average flow. Conclusion is that the balancing volume should be 4,9 times the average hourly flow.

With this demand pattern over the day the balancing volume is about 4,9/24 = 20% of the daily demand. As rule of thumb storage volume of 20 to 30% of the total daily demand is sufficient balancing volume for a predominant household demand pattern. Other patterns like industrial patterns will have another typical storage volume.

Apart from the actual balancing volume, a storage facility is also used as an emergency reservoir. If the pumping station for any reason is not capable of functioning properly, the supply can be done from the storage facility. The stored volume must bridge the time it takes to re-install the pumping station. This can vary from several hours to several days. When the emergency volume exceeds the balancing volume one must be extra alert on the actual flows and residence times in the storage.

5.6.2 Lay out of storage facilities.

Basically two types of storage are possible: high-level storage and low-level storage. A high level storage can be storage on a hillside or a traditional water tower. Main advantage of high-level storage is that the energy used to fill the storage is conserved as static energy. The storage will empty using gravity as driving force.

A low level storage must always be combined with a pumping facility to re-enter the water in the network. Obvious disadvantage is the energy waste in filling the storage, especially when this is in the middle of the distribution area.

Apart from the energy the types of storage differ in the way they are controlled. A high level reservoir acts as a very simple pressure monitoring point in the network. The level of storage is the actual pressure in the network at that point. The level can be monitored and when the level drops below a certain level, pumps can be switched on at the pumping station to fill the reservoir again.

A low level reservoir is more complicated, because both the incoming and the outgoing flow must be actively controlled. A throttling valve controls the incoming flow and pumps determine the outgoing flow. Figure 5.13 gives the pressure image of a high-level reservoir and figure 5.14 that of a low level reservoir.

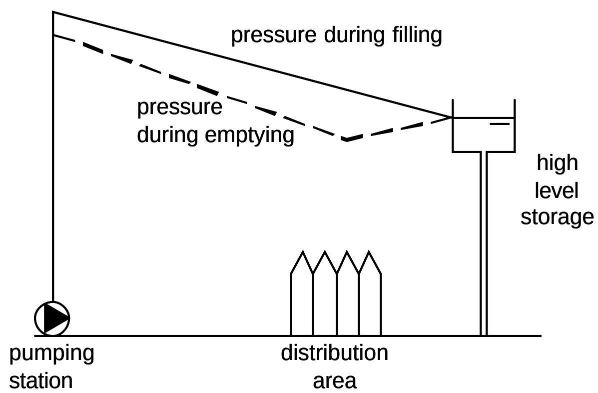

Fig. 5.13 - Pressure image high-level storage

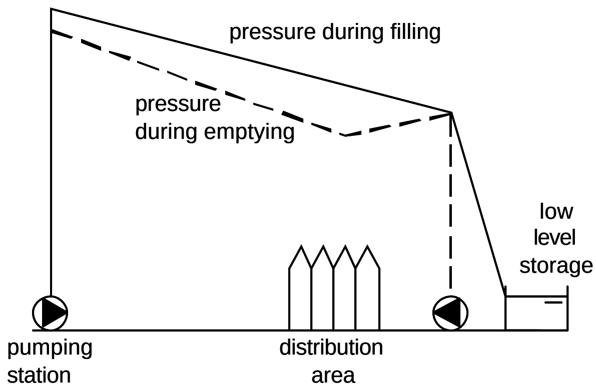

Fig. 5.14 - Pressure image low level storage

5.6.3 Storage in network calculation

Low level storage

Most simple way of modelling a low-level storage is to create a demand point with a demand pattern equal to the theoretical filling and emptying curve as derived from figure 5.12 and table 5.1. In the calculation this gives the desired behaviour of the storage volume, but the actual pumping station will differ from this ideal image.

A more complicated way is to model the complete pumping station and add a consumption node with the pattern of the inflow in the reservoir. During inflow conditions the pumps are out, but during outflow the pumps work and the outflow is dependent of the actual situation and interaction of the both pumping stations (the production plant and the reservoir pumps).

High level reservoir

A high level reservoir is a fixed pressure point in the calculation. This has the effect that the volume flow automatically is calculated to satisfy the pressure at that point. The pressure however is only fixed during the calculation of one particular point in time. When a time sequence is analysed and the dynamic behaviour of the reservoir is relevant, most calculation programs use a ‘ predictor-corrector’ algorithm. This means that a calculation is made with a reservoir level that is valid for the beginning of the time step. This results in a volume flow in or out of the reservoir. This flow is considered to be constant over the time step to be analysed, say 15 minutes. If this flow is constant during this 15 minutes, the level of the reservoir will change because of the in or out flow. This level is called the predictor. During the considered time step however the in or out flow will change. In case of an inflow the level in the reservoir will rise resulting in a higher pressure and causing the inflow to decrease. In case of an outflow the level of the reservoir will fall, so a lower pressure in the network and an increase of the volume flow.

If the volume flow is considered to be constant, the level in the reservoir will reach another value at the end of the time step. The beginning level and the end level are averaged and with this fixed pressure level the calculation is repeated, giving a more representative image of the flows and pressures. This is called the corrector step.

5.7 Setting up network calculations: data acquisition

5.7.1. Introduction

To perform network calculations as demonstrated in the forgoing paragraphs, a lot of data is required.

The modelling of a system is roughly divided in three stages: Setting up the model by gathering all hard and soft data, Calibrating the model using field data on pressures and volume flows and using the model in actual practise.

The data required can be divided in three categories:

- o Data on network feeds
- o Data on the pipes, nodes and geometry
- o Data on the demands

5.7.2 Data on network feeds

All the supply points should be modelled. Supply points are for instance pumping stations, booster pumping station, high-level reservoirs and low-level reservoirs with pumps. A supply point can be modelled as

- o A pumping curve or a combination of curves. This will simulate a pumping station and the steering rules. Mathematically this is a Q-H relation, which is iteratively solved.
- o A fixed pressure point. This type of modelling is used for a high level reservoir or a water tower. Mathematically this is a pressure boundary with a constant pressure; the volume flow is calculated.
- o A fixed supply modelled as a negative demand. Mathematically this is a fixed (negative) demand and the pressure is calculated.

Dependant on the goal of the calculation the type of modelling can be chosen. For instance a first calculation for design of a new transport line (see paragraph 5.3) a fixed pressure point will suffice. The desired capacity of the feeding point will be determined.

A fixed supply point can be used if a fixed amount of water will be supplied from one point, for instance a pumping station that will be used as a base line supply in combination with a more variable pumping station.

The most complicated modelling with a set of pumping curves with a steering hierarchy will be used to simulate the behaviour of an existing system.

Often the data on the supply must be collected manually.

5.7.3 Data on pipes, nodes and geometry

The bulk of the fixed data is on the geometry of the network. Lengths of pipe with the same characteristics are modelled from node to node. In the nodes the demands are concentrated.

Most of the water companies have automated mapping systems or network information systems that can easily produce the necessary information. A detailed information system will model every connection as a node with pipes in between them. This will result in a network in which every connection is represented by a node and pipe segments between the nodes have a typical average length of 10 to 15 meters. For a relative small town as Delft with 75000 inhabitants and about 33.000 connections a model with at least 33.000 nodes and about the same amount of pipes could be made. In the paragraph on skeletonisation this will be dealt with.

Extra attention must be paid to the hydraulic validity of the geometry of the network. The databases are the digital representation of older analogue maps. The conversion from the analogue system to the digital system has involved a considerable amount of manual works, so errors in the system are likely. In the calibration process this will come out properly.

5.7.4 Data on demands

Data on demands are available on several levels in a water company. The most basic data can be found in the billing system. Most water companies in the Netherlands have a fully metered system, so every connection has a known demand over an extended period of time, mostly one year. For calculations the demand situation on a 10 minutes or hourly base are necessary. Using a peak factor these data can be translated. Looking at an individual connection the peak factor for the average demand is very high: an individual uses about 120 to 150 litre per day, resulting in an average demand of 5 to 6 litre per hour. Using a kitchen tap causes a volume flow of 600 litre per hour, resulting in a peak factor of at least 100.

5.8 Skeletonisation

Building models from GIS systems is a very powerful way to constitute models of a network. There are two major drawbacks in pursuing this way:

- o The models consist of tens of thousands of nodes and pipes. This results in a slow working model with a lot of irrelevant information
- o The original GIS and billing systems are not set up for calculation purposes, but for registration purposes. The data in the system are not checked for hydraulic relevance and consistency.

Skeletonising is a way to reduce the number of pipes and nodes in a model without loosing hydraulic capabilities. The second reason for skeletonising is to check the hydraulis consistency of the data and to automatically improve the quality of the data. Result is that the number of nodes and pipes can be reduced by 80 till even 95% without loosing hydraulic relevance. This will save time in calculation, but more important is that interpretation of calculation results is more accurate.

Skeletonisation follows a set of rules, which are explained briefly.

The first steps are a hydraulic check of the system.

- o Adjacent similar pipes (equal diameter and k-value) are joint together. The connecting nodes are removed, while the possible demands are divided over the remaining nodes. If through joining a double pipe appears, the joining will not be done (see figures)

- o Nodes that are very close to each other, but are not connected will be joint if they are both not connected to more than one other node. The thought behind this is that in the digitalisation process the node is ‘clicked’ twice without the connecting pipe. This situation is typical for systems

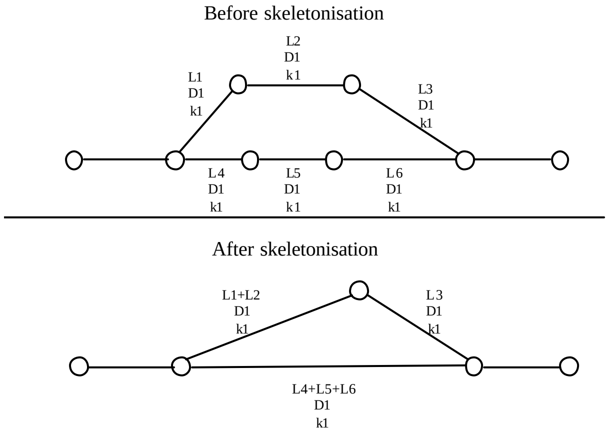

Fig. 5.15 - Picture before and after skeletonisation

that are converted manually from analogue maps to digital maps.

- o Nodes that are connected by short pipes (shorter than a predefined threshold) are joint together and for the pipe an hydraulic equivalent diameter at a standard, but recognisable, k-value (for instance 0,31) is calculated. The underlying thought is that the pipe has some repaired elements. For instance a Cats Iron pipe that is locally repaired or replaced by PVC pipes with slightly different diameters. Hydraulically the pipe can be considered as one.
- o Island identification
  Loose nodes are removed from the system. Also pipes and nodes that have no connection to a feeding point are removed from the model

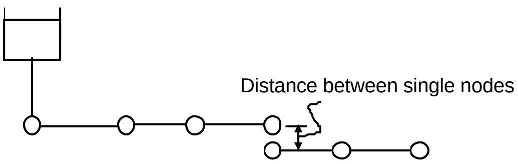

Fig. 5.16 - Picture distance between nodes

After the 'administrative' check of the model, the hydraulic skeletonisation can be done. For one particular demand situation the complete network is calculated. With the results the following steps are conducted:

- o Pipes with a diameter smaller than a certain threshold and a pressure gradient smaller than a predefined threshold are removed. Starting with a node towards which water flows (a commuting node or an end node) nodes and pipes are removed. Demand is moved to the upstream node.

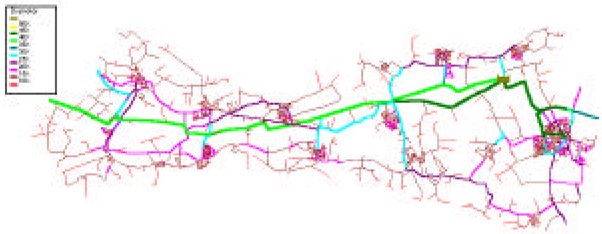

Fig. 5.17 - Unskeletonised network

- o End pipes below a threshold diameter with a small pressure gradient are removed. Demands are moved to the upstream node.

With these simple rules a lot of nodes and pipes will be removed, resulting in a workable network model.

Figure 5.17 and 5.18 show an unskeletonised network respectively a skeletonised network.

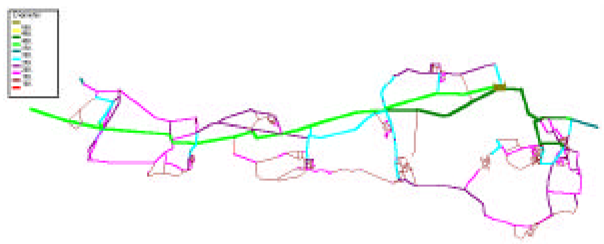

Fig. 5.18 - Skeletonised network