THESIS ON CIVIL ENGINEERING F41

Hydraulic Power Capacity of Water Supply Systems

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Declaration:
Hereby I declare that this doctoral thesis, my original investigation and achievement, submitted for the doctoral degree at Tallinn University of Technology, has not been submitted for any academic degree.

/Joonas Vaabel/

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INTRODUCTION

General

Although the gravity driven water distribution systems (WDSs) exist (especially in mountainous areas), most of the systems nowadays involve at least some degree of pumping in order to treat the raw water and guarantee adequate pressures at consumers' demand nodes. The development of automation and more reliable pumping systems has decreased the need for labour in water utilities perspective to maintain the WDSs. Therefore electricity costs for pumping comprise the major part of most system's operating budgets. The American Water Works Association Water Loss Control Committee indicates that 2 to 10% of the power consumed in a given country is by water utilities (Karney et al. 2009). This has lead researchers around the globe to find the best solutions in WDS design and rehabilitation in order to decrease the energy requirements for pumping.

However, the eager optimization of WDSs could lead us to the other extreme. The rapid development of cities and future uncertainties of water demand conditions could severely affect the reliability of WDSs. From a consumer's point of view it is natural to receive the water from tap regardless of the circumstances that might occur in the water distribution network (WDN) – leakages, pipe bursts, pump failures, etc. The systems that are well optimized for today's demand conditions could not cope with the increase of demand or unexpected failures in the WDN. The awareness of how vulnerable the WDSs could be in the eyes of terrorists has also pointed out the need to assess the reliability of WDSs. Both the energy reduction for pumping and maintaining the proper functioning of WDSs now and in the future has to be regarded in joint operation.

Objective of the Thesis

The main objective of the thesis is to introduce new capacity reliability measures in order to estimate the efficiency of WDNs and their ability to cope with sudden changes in everyday water demand, for example, in a fire flow situation.

The WDN is designed to serve customers considering several criteria - hydraulic reliability as well as good water quality. Hydraulic capacity and reliability are to guarantee necessary flow rate and pressure, however too low velocities are not good for the water quality in the network. This thesis covers the analysis of hydraulic reliability and efficiency of pumping stations.

The analysis could be applied for newly designed water distribution system as well as for rehabilitation of existing water networks.

The concept of hydraulic power for the analysis of water distribution network characteristics is described in detail in this thesis. The energetically maximum flows in individual pipes and in the WDN as a whole are determined. A capacity reliability indicator, called a surplus power factor*,* is introduced for individual transmission pipes and for the distribution network. The surplus power factor s characterizes in a sense the reliability of the hydraulic system, and can be used together with other developed measures to quantify the hydraulic reliability of water networks. The coefficient of the hydraulic efficiency η_n of a network is defined. An in-service WDS is analyzed to demonstrate the s and η_n values in a real water network at different demand conditions.

Layout of the Thesis

This thesis is divided into five chapters.

Chapter 1 reviews the literature covering studies from the early 1990s to the present. The objective is to focus on the studies mainly connected with WDS reliability. Also, studies on pumping efficiency are described in brief in order to connect the topic to the theoretical part of the thesis.

Chapter 2 presents the theory of the hydraulic power analysis for a WDS. This part describes the novelty of the current research: the capacity reliability index of the surplus power factor and the coefficient of network efficiency are developed.

Chapter 3 presents the development of the surplus power factor analysis for WDSs and the calculation algorithms.

Chapter 4 describes the case studies for in-service WDSs.

Chapter 5 summarizes the results and presents the conclusions and future research work.

Acknowledgments

I would like to express my deepest gratitude to my supervisor Prof. Tiit Koppel for his guidance and advice throughout the PhD studies.

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1. LITERATURE REVIEW

1.1. General

The literature review covers the studies conducted during the past couple of decades. It is the period when the reliability of WDSs has received considerable attention. This involves the development of the theory of hydraulic reliability and several reliability indexes of WDSs. As the theory of the optimization of WDSs had already been well covered, focus shifted towards the fact that “too optimized” WDNs already affected the overall reliability of systems. Therefore, the optimization process was integrated with the reliability indexes in order not to diminish the WDSs performance against unexpected failures.

Energy prices continue to grow and despite the rapid development of renewable energy sources, the reduction of energy needs for pumping is still a topical subject. However, until now water utilities had only the reduction of pumping cost in mind. The result was that many articles were published regarding pump scheduling in order to reduce pumping cost without actually decreasing the energy need for pumping, using time variably electricity tariffs. Now more attention is paid to actual energy reduction for pumping, which will decrease the carbon dioxide emissions.

1.2. Background of Water Distribution Modelling

Analysis of the quality, reliability or other aspects in WDSs is based on a calibrated model. The WDS hydraulic model can be linked with the external packages to solve the relevant problems. Today hydraulic modelling is tightly connected with the optimization process and the commercial software packages often have the ability to help engineers in the design process (as the design process itself is often "finding the best solution" problem).

The WDS modelling is out of scope of this thesis and the modelling processes are reviewed in brief (Figure 1.1). In the current thesis calibrated hydraulic models were used in order to analyze the reliability of WDNs and therefore in reference to the latter flowchart, focus is on the steps involving "using the model". Still, a thorough understanding of the behaviour of WDN models is needed in order to connect the hydraulic modelling with the WDS reliability analysis.

The complexity of the WDN optimization problem was explained in the analysis of Walski (2001). In most cases a simplistic procedure based on cost minimization is used. Therefore, in some cases the solutions for pipe diameters could be unrealistic, and a good engineer will never accept these results. Walski’s (2001) recommendation is to optimize a network considering the net benefit analysis, which is a multi-objective task. The most difficult problem for a designer is the future demand prediction that always involves a great deal of uncertainty (Walski, 2001).

The definition of the best design is difficult. All the design solutions, which satisfy flow and pressure requirements, are feasible designs (Sharma and Swamee, 2006).

Water distribution design problems cannot be solved by a single model, but rather different problems can be described as system master planning; transmission main preliminary design; subdivision layout; and rehabilitation of existing systems (Walski, 1995).

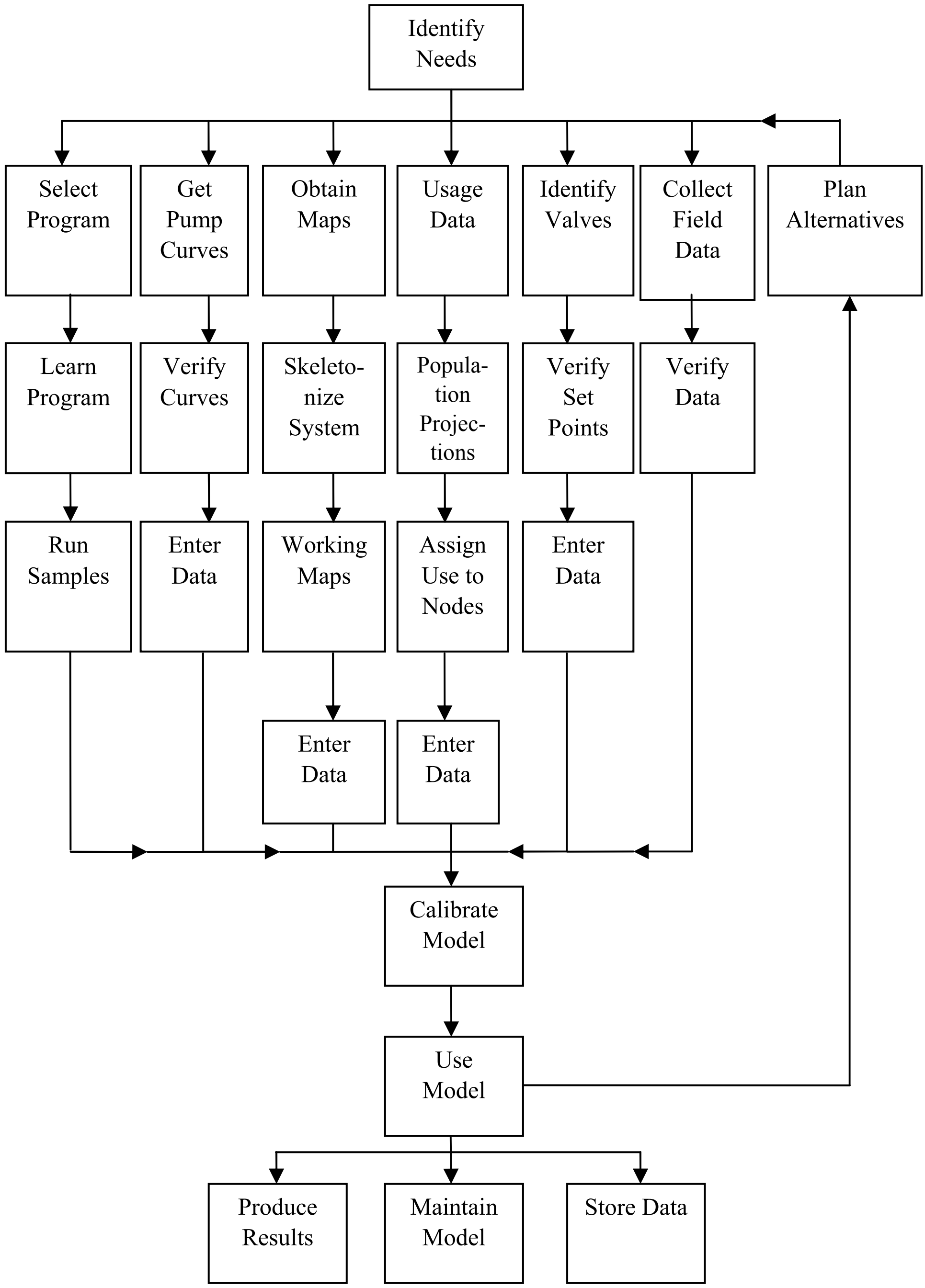

Figure 1.1 Flowchart of the modelling process [Adapted from Haestad et al. (2004)]

1.3. Water Systems Labyrinth

By the order of Canadian water companies and centres of excellence, Karney et al. (2009) compiled a comprehensive report on technologies or systems to reduce water and energy demand. The report by Karney et al. (2009) concerns particularly the water and energy that is associated with supplying water to urban area residents and industries. In order to map the huge amount of processes in WDS modelling, Karney et al. (2009) established a complicated diagram or "Labyrinth" (Figure 1.2). The figure displays the numerous processes, sub-processes, states of being with their associated causative factors, and interrelationships that are crucial to a distribution system's operation and performance. The figure shows that one main category is reserved for capacity, which plays an important role in order to supply consumers with sufficient flow and pressure. Karney et al. (2009) define the capacity by the sizing, construction, installation and configuration of the distribution system with all its elements and includes installed, reserve, treatment and pumping capacities. In a broader sense, capacity could also include natural water supplies, such as lakes, rivers, aquifers, etc (Karney et al., 2009). The current thesis studies only WDSs in cities (i.e. piping and pumping systems).

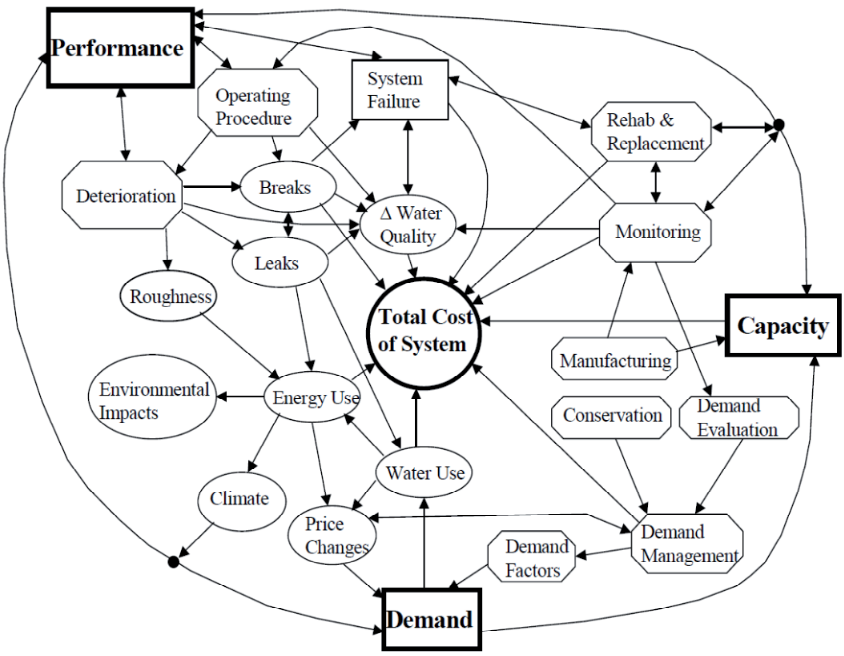

Figure 1.2 The labyrinth of water distribution systems [Adapted from Karney, Colombo and Deng (2009)]

1.3.1. Water-energy nexus

As could be seen from Figure 1.2, there is a direct link between water and energy use. Through the past centuries water has always been a source for energy. Hydropower has been harvested since ancient times and it has been a good and reliable energy source until industrialization in the 19-20th century created an energy consuming leap that introduced thermal power as a substitute for those greater demands.

Nowadays water has become more of an energy consuming agent. Despite hydropower is harvested even more than centuries back, still more energy is required in order to pump the water to large consumers, of which ironically energy production itself takes a big lump (as for cooling) while domestic water demand has a smaller role in overall water (and energy) consumption.

This creates an interesting relationship between water and energy and is widely known as water-energy nexus shown in Figure 1.3.

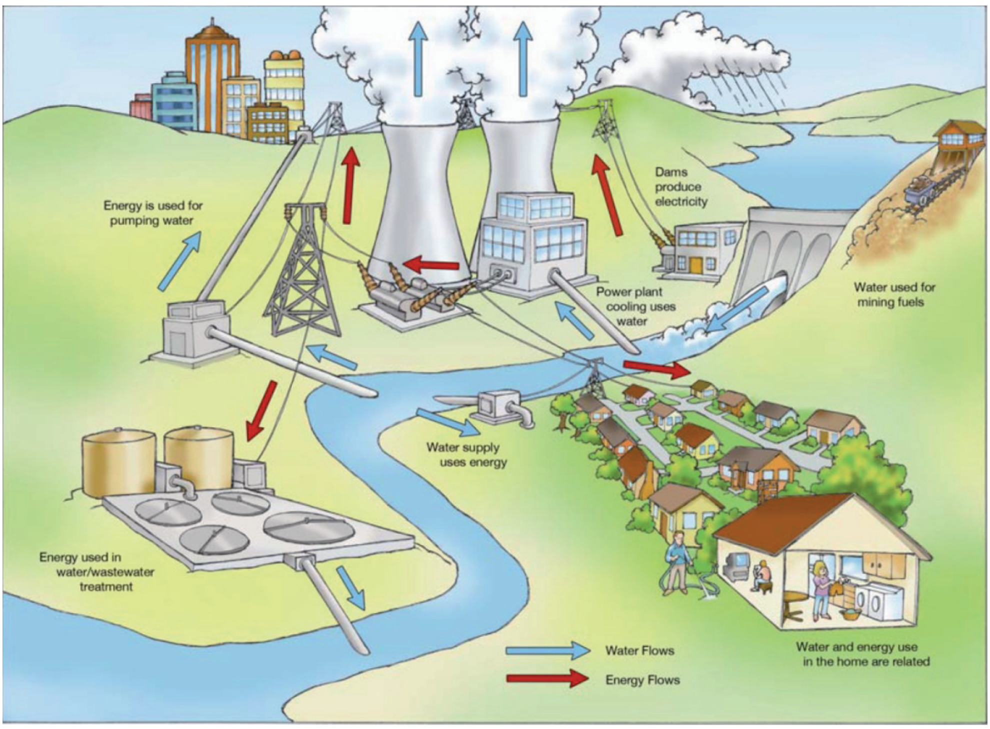

Figure 1.3 Interrelationships between water and energy [Adapted from U.S. DoE (2006)]

Technical report by Torcellini et al. (2003) has pointed out that domestic and commercial water consumption in the United States takes 12% of overall water withdrawal while agriculture and thermoelectric production has a large share, as shown in Figure 1.4.

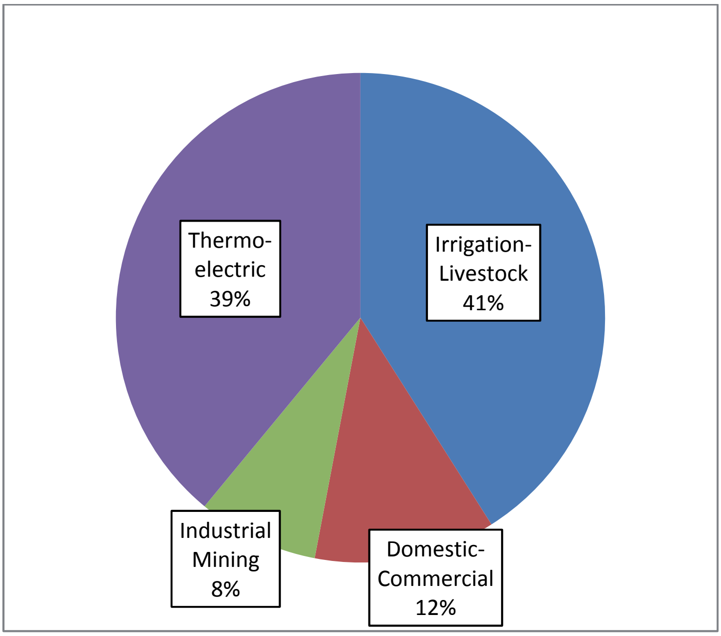

Figure 1.4 Percentage of total water withdrawals in the United States [Source: Torcellini et al. (2003)]

The current situation in Estonia varies a great deal from that described in the U.S. According to Estonian Statistical Agency (2010), the energy production itself is by far the most water demanding sector in the society, as shown in Figure 1.5. Since most of the electrical power is generated by thermoelectric plants, the energy production sector needs quite a large share of energy that it produces. However, overall water consumption differs country-by-country depending substantially on the climatic zone and topography.

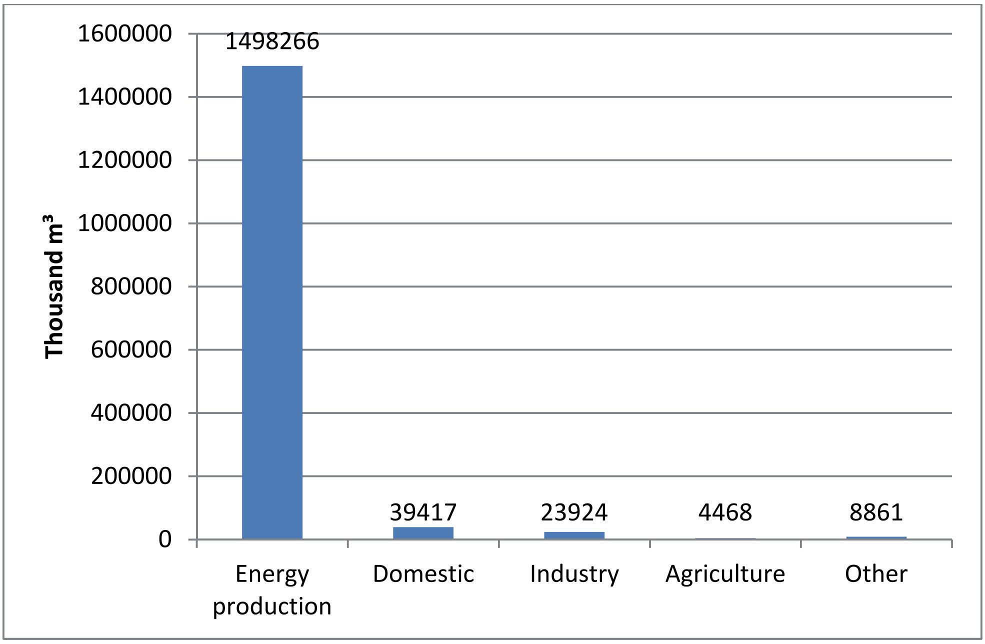

Figure 1.5 Water consumption in Estonia by sector [Source: Statistics of Estonia (2010)]

Although the share of residential consumption seems not to dictate the overall water needs, decrease in its value is still the interest of many researchers. The most interested parties are probably water companies. The cost for water treatment and pumping to consumers for 24 hours a day is the expenditure, the reduction of which even at small amounts could give good results at cheaper water prices for consumers or, more likely, higher profit to water companies.

1.3.2. Relating energy to system performance

Karney et al. (2009) clearly identify that the energy requirements associated with water supply emerge from the need to pump the water from its source to its destination. Hence, the same equations that are used for estimating the hydropower production can be used in order to estimate energy needs for pumping. The expression for the power required by a pump is related to the product of total dynamic head and the discharge through it, as represented in Eq. (1.1).

$$P = \frac{\gamma QH}{\eta}, \tag{1.1}$$

where γ is the specific weight of water; Q is the flow; H is the head and η is the pumping efficiency.

The energy requirement is defined as the power usage over a convenient time period and the discharge turns into the volume of a pumped flow. The energy requirement for pumping can therefore be defined as

$$E = \frac{\gamma V H}{\eta}, \tag{1.2}$$

where V is the volume of the pumped water.

1.4. Pump Head-Discharge Relationships

It is well known that the relationship between a pump head and a pump discharge is given in the form of the curve characteristic of a pump. In order to describe analytically the pump flow versus the head relationship, hydraulic modelling packages fit H=(Q) data points into the polynomial curve. A more common approach is to describe the curve by using a power function in the following form (Haestad et al., 2004):

$$H = H_c - gQ^m, \tag{1.3}$$

where H_c is a pump head at zero flow; Q is the pump flow; g and m are coefficients which determine the pump curve shape. In this case at least three H=(Q) data points need to be inserted - head at zero flow, design flow and maximum flow. Then the regression analysis is performed to determine the coefficients g and m, which is usually done by the hydraulic solver.

1.4.1. Fixed-speed pumps

The characteristic curve of WDSs does not necessarily meet the actual consumer requirements. Since the demand usually varies throughout the day it would yield also the pressure to change according to the pump's characteristic curve. In order to maintain the constant (or little variable) pressure in the system the demand variability is usually compensated with the use of water tanks or throttling/regulating valves. Operating the system by using throttling valves has been compared to driving a car with brakes on (Lingireddy and Wood, 1998). Fortunately with the replacement of old constant-speed pumps with the VFD pumps, this problem seems to have been removed.

1.4.2. Variable speed pumps

With advancements in variable frequency drives (VFD), variable-speed pumps became more common on the water distribution industry by replacing conventional fixed-speed pumps. At least all the new WDSs that are rehabilitated or designed in Estonia incorporate the use of variable-speed pumps. The additional cost of VFDs compared with fixed-speed pump without VFD has become marginal compared to today's construction cost of a pump.

Sarbu and Borza (1998) conducted a study in order to reduce energy consumption in WDSs in Romania. The common practice during the design process is slightly (or even more) oversize the pump to satisfy larger demands for future development areas. This leads to pumps duty points that are even outside of the pump diagram. Sarbu and Borza (1998) analyzed characteristics of various pumping stations and compared pumps with throttling valves and variable speed pumps. It was concluded that the method of pump regulation with variable speed drives is most for the functional optimization of pumps, as it correlates the pumped flow with the real water consumption.

Lingireddy and Wood (1998) presented several examples using a direct calculation algorithm (DCA) to evaluate improvements in the performance of WDSs utilizing variable-speed pumps. In addition to energy savings (economic benefits), there are a number of significant hydraulic advantages to using variable-speed pumps to control the operation of WDS. Lingireddy and Wood (1998) highlighted these hydraulic benefits as follows:

- Pressures can be maintained very close to the minimum required levels.
- Leakages directly related to pressure will be decreased.
- Times for pump operation can be more easily controlled and the off-peak pumping can be more efficiently used.
- Tank filling and draining rates can be controlled better in order to maintain water quality.
- A better response to fire events or pipe breakages is provided.
- Better transients control with starting or stopping pumps is ensured.
- Simple control of system flow rate is enabled.

However, improper selection of variable-speed pumps could cause noticeable drawbacks. Lingireddy and Wood (1998) referred to the following disadvantages by use of variable speed pumps:

- Variable speed pumps are less energy efficient when operating at lower efficiency speeds.

- Improved operations using variable speed pumps may result in less overhead storage and lower safety factor for fire protection. However, in terms of water quality, large overhead storage results in lower water quality.

1.5. Pumping Efficiency Analysis

A key interest of hydraulic systems designers and scientists has always been pumping efficiency. Below the development of pumping efficiency is briefly reviewed starting from some early 1990s studies, since a complete study is out of scope of this thesis.

Since the largest operating costs for water utilities are the cost of energy to run the pumps, even small savings on the reduction of pumping energy would yield a huge amount of finance for a year. Common operational problems that contribute to high energy usage are (Haestad Methods et al., 2004):

- Pumps that are no longer pumping against the head for which they were designed.
- Pumps which were selected based upon a certain cycle time and are being run continuously.
- Variable-speed pumps being run at speeds that correspond to inefficient operating points.

In the pumping efficiency analysis one has to consider that during the diurnal demand variation the pump(s) have several operating points that will cause the efficiency to vary along with the demand conditions change.

Tarquin and Dowdy (1989) stated two major reasons why considerable energy conservation may be possible in municipal water distribution systems if the pumps in the WDS run at constant speed. The assumptions may not be relevant for newly developed WDSs, but could be applied to systems that have been in operation for a long time. Firstly, Tarquin and Dowdy (1989) pointed out that WDSs are usually created by step-by-step expansion projects as newly developed areas are linked with the existing system. In the design process, such practices take into account only the system currently under expansion, with little consideration given to the optimization of the system as a whole. Secondly, the centrifugal pumps remain in service for several decades and poor maintenance and normal wear decrease the efficiencies of the pumps to a point where different operating procedures could have effect on pumping costs (Tarquin and Dowdy, 1989). Although variable speed pumps and real time demand control are used in everyday practice nowadays, the concepts of Tarquin and Dowdy (1989) are welcome in order to start the WDS optimization from the right point.

Brion and Mays (1991) clearly stated the objectives of optimal operation of pumping stations. The improvement of pump operation efficiency focuses on three different aspects: inefficient pump combinations, inefficient pump scheduling, and inefficient pumps. The objective function is to minimize pumping cost over a planning horizon considering hydraulics involved in the WDS, bound constraints on decision variables, and other constraints that may reflect operator preferences or system limitations. The optimal pump operation problem is a large-scale nonlinear programming (NLP) problem, which required some special techniques in the early 1990s to overcome the small computational power (compared to nowadays). In their study Brion and Mays (1991) used linked approach between hydraulic simulation and optimization approach in order to reduce the steps of the solution process.

Energetically optimal flows in an individual pipe and in a series of pipelines with discrete and continuous distributions of water consumption with time variability are discussed by Ainola et al. (2003a). Energetically optimal head distribution in pipes is determined, and the results help to increase the efficiency of energy consumption by the rehabilitation of water distribution networks.

Ormsbee et al. (1989) analyzed the optimization of pumping costs. It was noted that although the total energy consumption charges associated with a pump operation can be decreased by improving the efficiency of individual pumps or pump groups, such measures have little impact on reducing the costs with time-of-day energy rate schedules. Therefore it is required to modify the pump operation procedures during the time. Ormsbee et al. (1989) developed an optimal pump operation methodology by composing two basic phases: the development of an optimal tank trajectory and the development of an optimal pump operating policy.

Maksimović and Masry (2008) studied an alternative design called ESC (Energy Saving Concept) which considered pumping to lower reservoir elevations with subsequent boosting to low pressures in order to reduce the risks of pipe failure and leakages, as well as pumping costs. The case study carried out by Maksimović and Masry (2008) showed that the use of boosting pump in a system with low tower elevation proved to be up to 19% more cost-effective than using PRVs in systems with high tower elevations.

1.5.1. Developing a curve relating flow to efficiency

In order to analyze the efficiency of pumps during the diurnal demand conditions the easiest way is to convert pump efficiency versus flow data into some kind of mathematical equation. In general, a curve representing the relationship between pump discharge (Q) and pump efficiency (η) can usually be described by the equation for an inverted parabola, as shown in Eq. (1.4) (Haestad et al., 2004)

$$\eta = \alpha_0 + \alpha_1 Q + \alpha_2 Q^2. \tag{1.4}$$

Cubic polynomials could be used in order increase the accuracy of the efficiency curve. The easiest way to find the values for coefficients α_0, α_1 and α_2 is with spreadsheet programs by inserting at least three data points and developing the trend line curve with the equation.

For example, Table 1.1 shows three sample data points that illustrate the flow versus the efficiency of a pump.

Table 1.1 Flow versus efficiency data points

Q (L/s)	η (%)
100	55
200	85
300	60

Respective curve according to the data above is illustrated in Figure 1.6. Also, as described in Eq. (1.4), the coefficients for the polynomial function are created using a spreadsheet trend line feature.

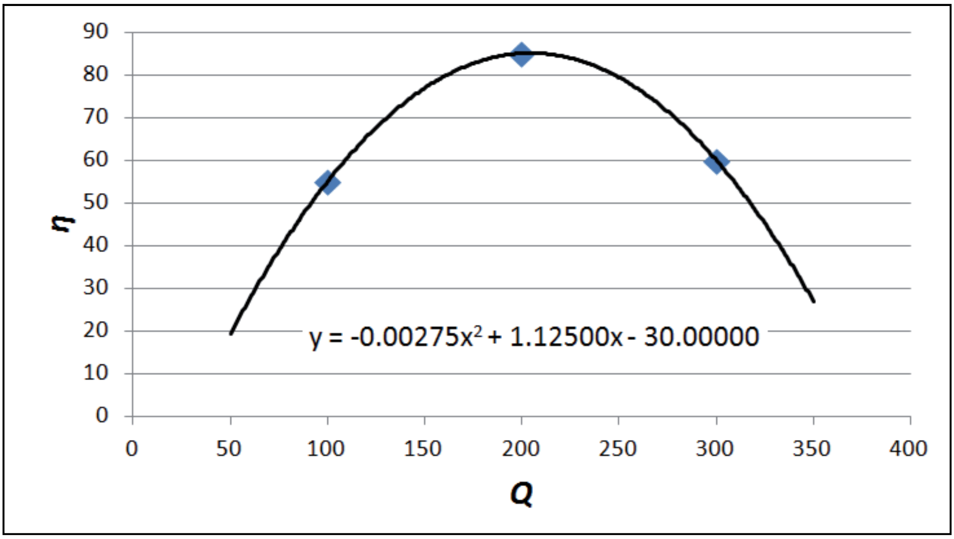

Figure 1.6 Flow versus efficiency curve

In order to develop the flow versus efficiency curve for variable-speed pumps, the total dynamic head has to be considered. In general, according to Haestad et al. (2004), the equation can be approximated by

$$\eta = \alpha_0 + \alpha_1\left(\frac{Q}{n}\right) + \alpha_2\left(\frac{Q}{n}\right)^2, \tag{1.5}$$

where n is the ratio of pump speed/pump test speed.

Ulanicki et al. (2008) have developed mathematical formulation of pump efficiency curves that will avoid singularity (division by zero) when quadratic polynomials are used for an analysis. They found that for individual pumps, hydraulic characteristic curves are usually approximated by the quadratic polynomial or by the power law. By utilizing both approaches, Ulanicki et al. (2008) used real pump characteristics and compared them with analytical results that were developed by the proposed method.

1.5.2. Pumping scheduling

In their study, McCormick and Powell (2003) proposed a progressive mixed integer heuristic which can provide discrete pumping schedules for large networks. Their approach contradicted the drawbacks of some pump scheduling methods where results were continuous, but discrete schedules were needed.

Borzi et al. (2008) developed a methodology for the evaluation of optimal pumping system operation through synthetic indices in Italian small WDSs. The objective was to minimize pumping cost based on different energy tariffs (i.e. cheaper, off-peak night hours). Based on Monte Carlo generation of many pumping schemes (2500 different pumping systems), Borzi et al. (2008) proposed the events when certain synthetic index values should be applied, i.e. when the index exceeds or is less than a certain value corresponding to energy rates will be more convenient to use.

Bunn (2006) conducted a case study in four U.S. cities in order to reduce energy costs for pumping. The developed software was designed to seek cost reductions in production costs as well as energy costs, however energy cost was dominating. According to Bunn (2006), to reduce energy cost the software seeks savings in three main ways:

- moving energy use into cheaper tariff periods, using water tower storage to supply customers;
- reducing peaks demand charges by limiting the maximum number of pumps these times;
- reducing energy required to deliver water in distribution through running a pump or group of pumps closer to their optimal efficiency.

1.5.3. Accounting carbon emissions

The pump scheduling by using different energy tariffs and using pumping into storage can reduce pumping costs. However, there is usually no energy reduction used for pumping. This fact has gained great attention during recent years since there is a worldwide target to reduce greenhouse gases (GHGs). While cost saving through time-of-use scheduling may seem attractive for water utilities, reducing the pumping cost does not necessarily reduce the pumping energy and therefore emissions of GHGs.

Wu et al. (2008) developed a new paradigm for the design of WDSs where minimisation of the costs of GHG emissions is incorporated into the optimisation of WDSs either as one part of the objective or as a second objective. Wu et al. (2008) used a discount rate of 1.4% compared to 8% to carry out the present value analysis (PVA), which enables future values to be translated to the present. The objective function process evaluation is illustrated in Figure 1.7. In the study by Wu et al. (2008), the emissions from the pipe manufacture were calculated using embodied energy analysis (EEA) and emission factor analysis (EFA).

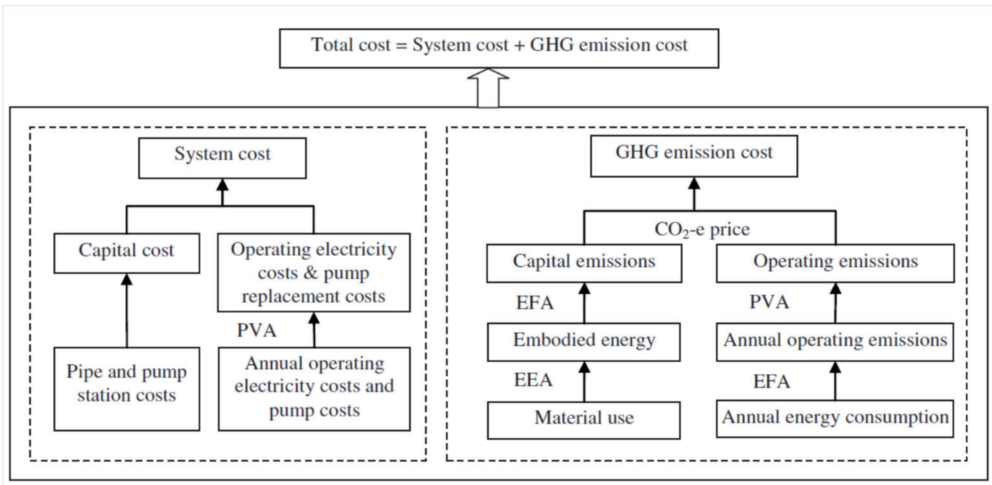

Figure 1.7 Objective function evaluation for the system and the GHG emission cost [Adapted from Wu et al. (2010)]

Reynolds and Bunn (2010) advanced the study (Bunn, 2006), highlighting the issue of reducing the energy cost as well as carbon emissions by pump operation more efficiently. The idea behind the energy savings is real time monitoring of WDS via SCADA system and operating the pumps according to water demand and pressure changes in the system. According to Reynolds and Bunn (2010), they achieved an average energy efficiency gain of a pump from 6% to 8.4% with few individual pump stations showing improvements greater than 20%.

Now several software packages are available, for example, WaterGEMS by Bentley that incorporate pumping scheduling optimization inside hydraulic modelling software using genetic algorithms (GA).

1.6. The Concept of Hydraulic Power

The concept of hydraulic power was used to assess the hydraulic reliability of a WDN by Park et al. (1998). The performance of a WDN depends on the ability of the network to meet the demand for the volume and the pressure of the flow. To evaluate the network simultaneously on the basis of both the flow and the pressure, the concepts of hydraulic power and energy transmission can be applied. In this research, two main characteristic values for a WDN, the flow rate and the pressure, were combined in a single dimension of the hydraulic power and the requirements for the hydraulic power were adequate to be incorporated in the reliability models (Park et al. 1998). This approach allows us to look at a feasible flow of the hydraulic power in a single pipe or in the WDNs and to analyze the hydraulic reliability of the system (Park et al. 1998).

The study was carried out with both Darcy-Weisbach and Hazen-Williams friction loss equations. For an individual pipe it indicated that the maximum hydraulic power delivered to the outlet of a pipe occurs when the frictional loss (by the Darcy-Weisbach eq.) is one third of the total head that is supplied at the inlet. When using Hazen-Williams equation, maximum power transmission occurs when the head loss is ~35% (Park et al. 1998).

Schneiter et al. (1996) used the concept of hydraulic power capacity to identify the pipes which by improvement would contribute most to the increase in the hydraulic capacity of the network. Their analysis for rehabilitation is based on a capacity-versus-cost trade-off curve, i.e., the volume flow was used alone for their rehabilitation estimates.

Schneiter et al. (1996) investigated the highest carrying capacity that a network can supply on a simplified model of WDNs that ignores the head conservation.

1.7. Energy Audit of Water Networks

A comprehensive energy analysis of the WDNs has been made by Cabrera et al. (2010). The usage of energy injected to the WDN was assessed, with emphasis on the energy losses resulting from the leakages. The energy lost by the leakages is divided into two parts: firstly, the increase of the friction losses dependent on the leakages and secondly, the lost water volume by the leakages. The energy audit is based on the power balance in the network, where a control volume principle is used for the system (Cabrera et al., 2010). In Figure 1.8 it is represented by the control volume and the incoming and outgoing flows of energy.

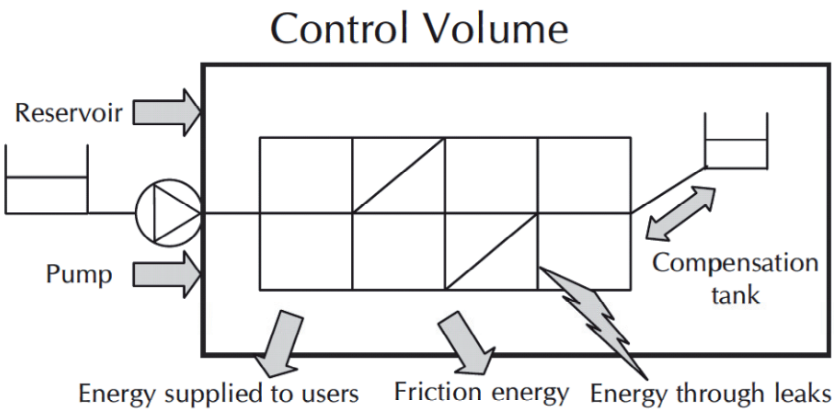

Figure 1.8 Water network as a control volume with the terms of the energy balance [Adapted from Cabrera et al. (2010)]

Cabrera et al. (2010) stated that the power supplied to the network is equal to the power delivered to the users plus the power losses of leakage and mechanical friction:

$$P_N + P_P = P_U + P_L + P_F \pm P_C \tag{1.6}$$

where P_N is the natural power supplied by reservoirs and tanks; P_P is the power supplied by pumps; P_U is the useful power delivered to users; P_L is the power loss due to leakages; P_F is the power losses due to friction and P_C is the change with time (negative or positive) of the potential energy in the tanks.
When integrating Eq. (1.6) over time, the power terms convert into energy terms.

As a result, Cabrera et al. (2010) summarized the energies resulting from the integration of input and consumed powers for a simulation period time as follows:

- Natural energy (supplied by external resources), E_N
- Shaft energy (supplied by pumps) , E_P
- Useful energy delivered to users, E_U
- Leakage energy losses, E_L
- Friction energy losses, E_F
- Compensation energy (associated with internal system tanks), E_C

1.7.1. Energy indicators

In order to characterize the efficiency of the WDS Cabrera et al. (2010) proposed five performance indicators as follows:

- Ratio between the real energy entering the system and the minimum useful energy, I_1
- Measure of the efficiency of the use of the energy injected to the system (which fraction of the total energy input is useful), I_2
- Hydraulic capacity of the network, I_3
- Energy loss due to leakage, I_4
- Ratio between the energy delivered to users and the minimum required useful energy, I_5

The value range for performance indicators follows as (Hernández et al., 2010):

- $I_1 \geq 1$. Preferably closer to one.
- $0 \leq I_2 \leq 1$. Preferably closer to one.
- $0 \leq I_3 \leq 1$. Preferably closer to zero.
- $0 \leq I_4 \leq 1$. Preferably closer to zero.
- $I_5 \geq 1$. Preferably closer to one.

It is noted here that I_3 can be brought to values close to zero, though eliminating friction losses requires large and costly piping and the water quality could be severely affected. If $I_5 < 1$, then average pressure levels are insufficient and below standards (Cabrera et al., 2010)

1.7.2. Energy audit case studies

As the energy audit concept is fairly new, only few case studies have been widely published.

Hernández et al. (2010) conducted the energy audit on a real WDS supplying around 100000 inhabitants. Daily and monthly analyses were performed for real and ideal (leak free) networks. As a result, Hernández et al. (2010) showed the energy requirement by performance indicators (section 1.7.1) that were the basis for the cost-benefit analysis.

1.8. Reliability of Water Distribution Systems

In order to satisfy a customer with good-quality water, the WDS must be designed to accommodate a range of expected emergency loading conditions. Lansey et al. (2004) classified these emergency conditions as follows:

- Broken pipes
- Fire demands
- Pump failure
- Power outages
- Control valve failure
- Insufficient storage capability

Reliability in these circumstances is usually defined as the probability that a system performs its mission within specified limits for a given period of time in a specified environment (Lansey et al., 2004). If the systems had to be optimized against the minimum cost, the abovementioned emergency conditions should have be considered.

To define the system reliability, Gessler and Walski (1985) developed the WADISO module composed of three major parts. The first, known as the simulation part, computes the pressure and flow distribution in pipe networks. The second part calculates the cost and pressure distribution for a set of user selected pipe sizes and changes the sizes within user specified limits until it finds the most economical arrangement which meets the pressure requirement. The third part considers fluctuating tank water levels and varying demand patterns and runs extended period simulation to the system.

Bao and Mays (1990) developed an algorithm to compute the system reliability, the methodology of which was based upon a Monte Carlo simulation consisting of three major components, i.e., random number generation, hydraulic simulator, and reliability computation.

Several papers of Duan et al. (1990a) focus on the reliability aspects for pumping stations. They analysed modified frequency and duration (FD) in terms of mechanical failure of the pump station and tanks and hydraulic failure in the system in order to make reliability analysis more realistic and complete. Duan and Mays (1990b) concluded that minimum-cost WDSs obtained from conventional optimization techniques cannot always guarantee a reliable system since these techniques do not emphasise or even consider the number of pumps and tanks in the system. Duan and Mays (1990b) stated that the number of pumps in the pumping station has a significant effect on the reliability-based optimal design of WDSs, which also verifies the need to model the pumps and tanks properly.

Goulter and Bouchart (1990) incorporated reliability measures into least-cost optimization design models for looped WDSs. "The probability of node failure" (PNF) measure was chosen in order to demonstrate the network reliability.

According to Cullinane et al. (1992), reliability to withstand emergency conditions, as mentioned previously, is usually incorporated in design standards and therefore it is assumed that the system will meet the demand and pressure requirements 100%. For example, in order to cope with pump failure, the design codes require usually backup pumps in municipal water supply systems. The same is true for fire demands - the system must meet the capability to provide adequate fire flows on top of everyday water needs. Hence, Cullinane et al. (1992) pointed out that most components in the WDS are repairable and availability rather than reliability concepts are more appropriate to evaluate WDSs.

1.8.1. Hydraulic availability

Cullinane et al. (1992) proposed a number of different measures of reliability indices, each of which could be useful, depending on the purpose of the analysis.

Cullinane et al. (1992) defined WDS hydraulic reliability as "the ability of the system to provide service with an acceptable level of interruption in spite of abnormal conditions". The availability specifies the percentage of time that the demand can be supplied at or above the required pressure. Cullinane et al. (1992) developed a continuous "fuzzy" system availability index function that illustrates the availability index versus the system pressure, as shown in Figure 1.9.

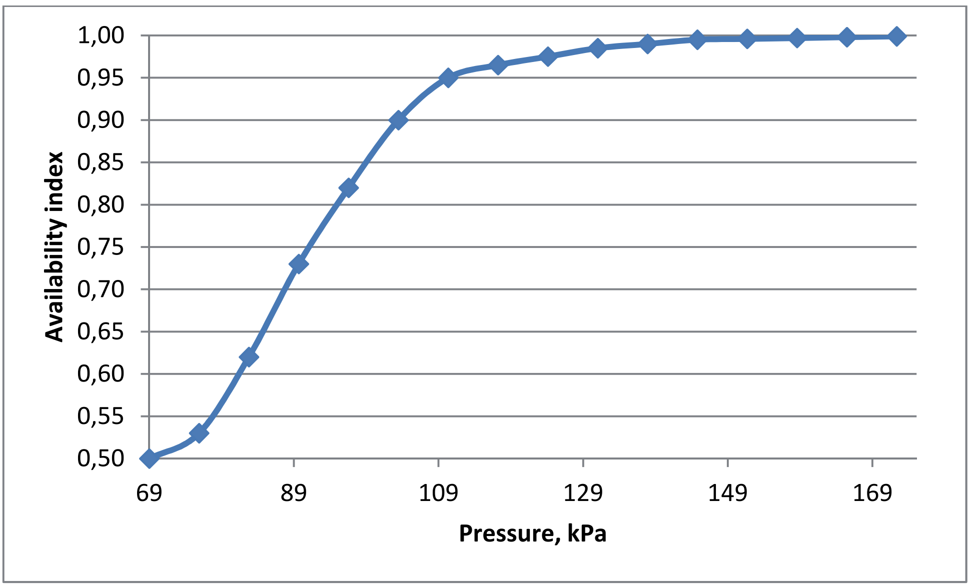

Figure 1.9 System Availability Index Function [Adapted from Cullinane et al. (1992), original pressure (psi) units were converted to kPa]

Shinstine et al. (2002) applied the approach developed by Cullinane et al. (1992) in a series of case studies in order to simulate the availability function in real WDNs. The results were plotted on the hydraulic availability versus the pressure curves similarly to Figure 1.9. The shape of the curve depends on the selected mean nodal pressure and the standard deviation of pressure.

In their work Lansey et al. (2004) summarized the reliability indices as follows:

- *Reliability* is the ability of the system to meet demand under a defined set of contingencies.
- *Availability* is the probability at a given moment that the system will be found in a state such demand does not exceed available supply or capacity.
- *Average availability* is the mean probability over a period of time being found in such a state.
- *Severity indices* describe the size of failures.
- *Frequency and duration indices* indicate how often failures occur and how long they last.
- *Economic indices* refer to financial consequences of shortages, also referred to as *vulnerability*.

Xu and Goulter (1999) used the FORM approach [also known as the advanced first-order second-moment (AFOSM)] in developing a WDN optimization model where inherent uncertainty in the nodal demands and the values of pipe coefficients, as well as the impacts of component failures were considered.

Ostfeld and Shamir (1996) and Ostfeld et al. (2002) applied the reliability analysis to multiquality water distribution systems (MWDS). According to Ostfeld et al. (2002), traditionally, reliability had been defined by heuristic guidelines, like providing water to consumers from looped networks (two alternative paths), or having all pipe diameters greater than a minimum prescribed value. However, in such manner the level of reliability provided is not quantified or measured. Hence, Ostfeld et al. (2002) developed a model that implements three basic steps:

- The definition of reliability measures ("single" and "multiquality")
- Inclusion of the stochastic nature of performance of each system component, and the consumers demands
- Compiling the previous two steps into single framework that generates random events

Cargano and Pianese (2000) substituted the hydraulic reliability index with the overall reliability index, which was defined as the weighted probability that the network will be able to satisfy user demand in fully or partially operational condition as a result of failure in one or more system components. In their case study, Cargano and Pianese (2000) emphasized the impact of diurnal demand variability on the load conditions for system components.

Kapelan et al. (2005) developed a robust WDS design methodology in order to overcome the uncertainties of future WDS characteristics. Kapelan et al. (2005)

defined the robustness as the probability that all network nodes are simultaneously equal or above the minimum requirements for that node. Nondominated sorting genetic algorithm II (NSGAII) and newly developed RNSGAII were tested on a number of different cases for the New York Tunnels reinforcement problem by Kapelan et al. (2005).

Martinez (2007) introduced an improved formulation that explicitly took cost consequences of pipe failures into account in the objective function. It was demonstrated by Martinez (2007) that the looped network can be less costly than a branched one if the reliability is considered in the optimal design of WDNs.

Brown (2010) advocated a new paradigm for water resources design - the end of reliability. The traditional approach is to design a system to "not fail" up to some reliability and then not to consider what happens when an exceeding event does occur. However, when designing a system up to certain reliability, e.g. one in hundred events, there will be failures already accounted that could be unacceptable in some cases (Brown, 2010).

In some countries, for example Netherlands, minimum reliability levels are set by government legislation - in case of failure of one element of the WDS the remaining supply capacity should be at least at a certain level of the maximum daily demand. Following this, the pressure dependant demand approach could be applied to WDSs, as seen in Figure 1.10. In reality it means that in case of some failure and pressure drop in the WDS, consumers at higher levels (for example, on the 2nd floor and higher) will not receive enough pressure to use a tap and therefore the overall demand condition in a WDS will change.

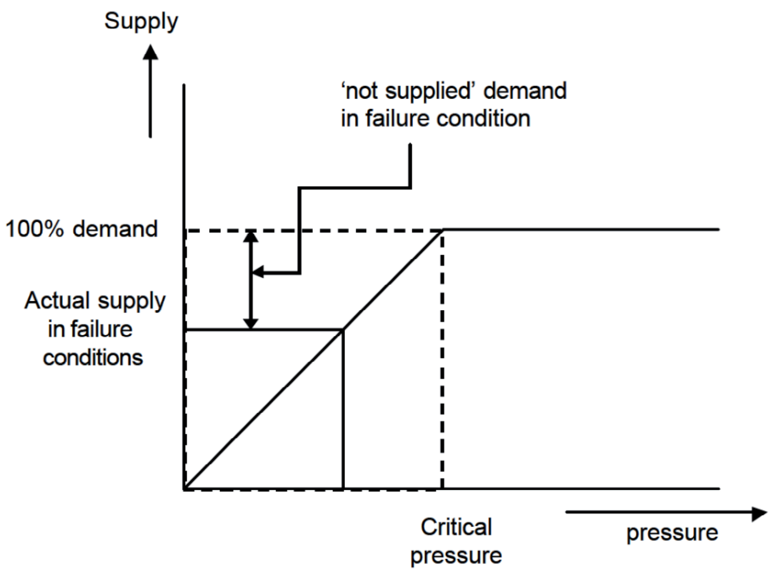

Figure 1.10 Pressure dependant demand [Adapted from "Reliability of Drinking Water Systems" (ocw.tudelft.nl)]

1.9. Resilience

The word "resilience" can have quite many definitions depending on the subject considered. In WDSs it is most widely regarded as a risk management tool against probable failure events that could affect the consumers to receive the water with required quality and pressure.

In terms of environmental management, Blackmore and Plant (2008) defined the resilience as follows (Table 1.2):

Table 1.2 Definitions of resilience and related concepts [as in Blackmore and Plant (2008)]

Concept	Definition	Assumptions and objectives
Engineering resilience	Return time to steady state following a perturbation	• Efficiency, constancy, and predictability • Single static stability domain
Ecological resilience	Magnitude of disturbance that can be adsorbed before the system redefines its functional structure by changing the variables and processes that control behaviour	• Persistence, change, unpredictability • Multiple static steady states
Adaptability adaptive capacity	Capacity of actors in the system to influence resilience	• Persistence, change, unpredictability, and slowly changing variables
Resilience	Capacity of a system to absorb disturbance and reorganize when undergoing a change so as to still retain essentially the same function, structure, identity, and feedbacks	• Multiple dynamic steady states
Transformability	Capacity to create a fundamentally new system when ecological, economic, or social structures make the existing system untenable	

In their risk management and resilience study, Blackmore and Plant (2008) proposed an approach to sustainable urban water systems involving the design and

implementation of integrated urban water systems (IUWSs). In order for an IUWS to be sustainable, its design needs to consider the threats from natural hazards, malfunctioning, misuse, operational failure, etc.
As shown in Table 1.2, the resilience concepts could be easily applied to WDSs as a tool in order to withstand certain probable and unwanted events.

Wang and Blackmore (2009) reviewed several resilience concepts and defined that in general, three aspects of resilience are considered in water resources systems:

- resilience against crossing a system performance boundary conditions
- resilience for system response and recovery after negative impacts
- resilience for adaptive capacity and management (Figure 1.11)

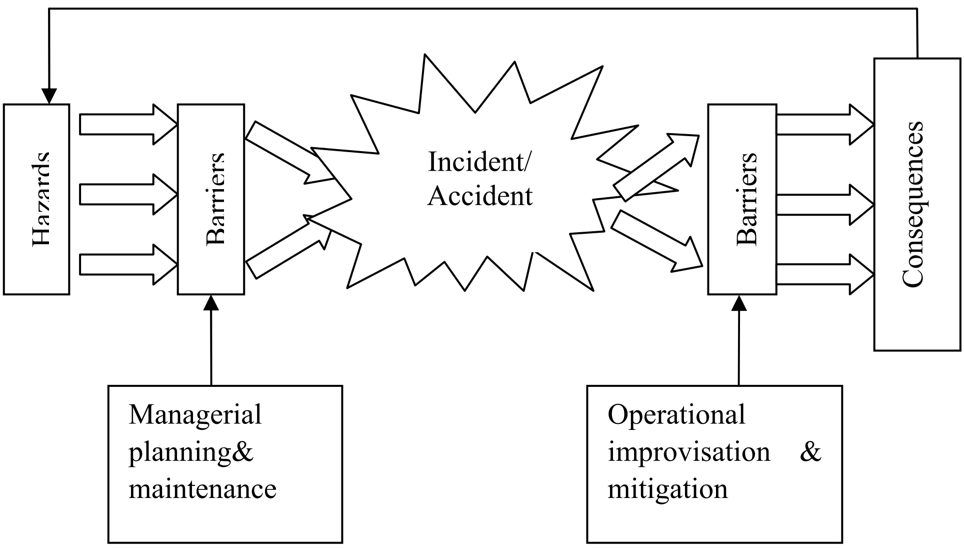

Figure 1.11 Schematic representation of system adaptive capacity/management [Adapted from Wang and Blackmore (2009)]

1.9.1. Water networks resilience

The optimization of WDNs has attracted researchers' interest for a long time and several approaches have been analyzed to minimise the cost of the system. In terms of the minimum cost function, a tree shaped network provides the most optimum layout of piping between a source and consumers. However, in these solutions the reliability concept is not incorporated adequately and are therefore avoided in everyday practice.

1.9.2. Resilience indexes

In order to increase the WDN's flexibility against sudden failures, the system should have some resilience that will smoothen the possible pressure drops or lack of water even in an unforeseen situation. The result is over dimensioning of the system. In order to describe the situation how resilient the system is compared to the minimum requirement, several authors have proposed to use resilience indexes that will allow the determination of the WDN's flexibility against unforeseen conditions.

Todini's resilience index

In order to describe the reliability in looped systems, the resilience index (I_r) was introduced by Todini (2000). The whole idea for looped systems is to provide more power than required at each node, in order to have some resilience in case of failures in the system. Hence Todini (2000) defined the resilience index as the ratio of the surplus internal power in the network to the maximum power that could be dissipated internally, after satisfying the constraints in terms of demand and head at the nodes

$$I_r = 1 - \frac{P_{\text{int}}}{P_{\text{int,max}}} \tag{1.7}$$

where P_{int} is the amount of power dissipated internally in the network and $P_{\text{int,max}}$ is the maximum power that could be dissipated internally in order to satisfy the constraints in terms of nodal demands and the nodal heads.

$$P_{\text{int}} = P_{tot} - \gamma \sum_{i=1}^{nn} Q_j^{req} H_j \tag{1.8}$$

where Q_j^{req} is the demand at a node j; H_j is the head at the node j; γ is the specific weight of water and nn is the number of nodes.

P_{tot} is the total power available at the entrance of the water distribution network given as follows:

$$P_{tot} = \gamma \sum_{r=1}^{nr} Q_r H_r \tag{1.9}$$

where Q_r is the discharge delivered by the reservoir r; H_r is the head at the reservoir r and nr is the number of reservoirs feeding the network. $P_{int.max}$ is calculated as follows:

$$P_{int,max} = P_{tot} - \gamma \sum_{i=1}^{nn} Q_j^{req} H_{min,j} \tag{1.10}$$

where $H_{min,j}$ is the minimum required head at the node j at which the nodal demands are to be supplied.

Substituting the values of P_{int} and $P_{int,max}$, the resilience index can be written as [Jayaram and Srinivasan (2008)]:

$$I_r = \frac{\sum_{j=1}^{nn} Q_j^{req} \left(H_j - H_{min,j} \right)}{\sum_{r=1}^{nr} Q_r H_r - \sum_{j=1}^{nn} Q_j^{req} H_{min,j}} \tag{1.11}$$

Todini (2000) conducted two case studies (two-loop network and classical main loop network) and plotted the Pareto set of solutions where the cost was compared against the resilience index. It was discovered that in the first part of the Pareto limiting curve, the resilience index could be doubled with very small increases in the cost, as seen in Figure 1.12.

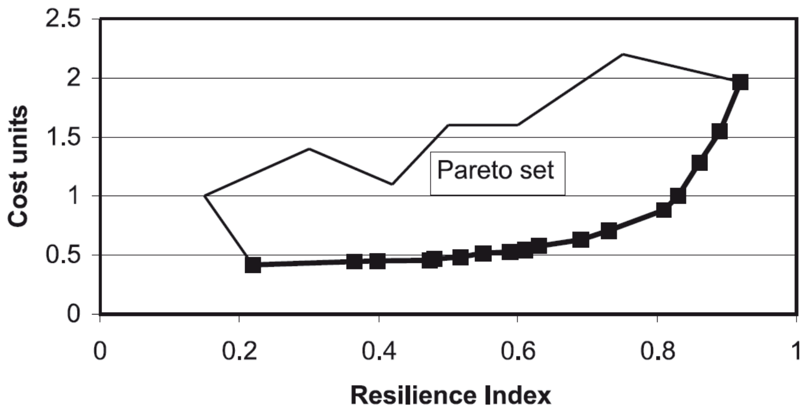

Figure 1.12 The limiting curve of the Pareto set of solutions in the two objective spaces [Adapted from Todini (2000)]

Also, along with the resilience index, Todini (2000) developed a failure index and an available surplus head index. The first of which [as seen in (Figure 1.13)] identifies infeasibilities during the optimisation process and the latter one gives the surplus head at each node.

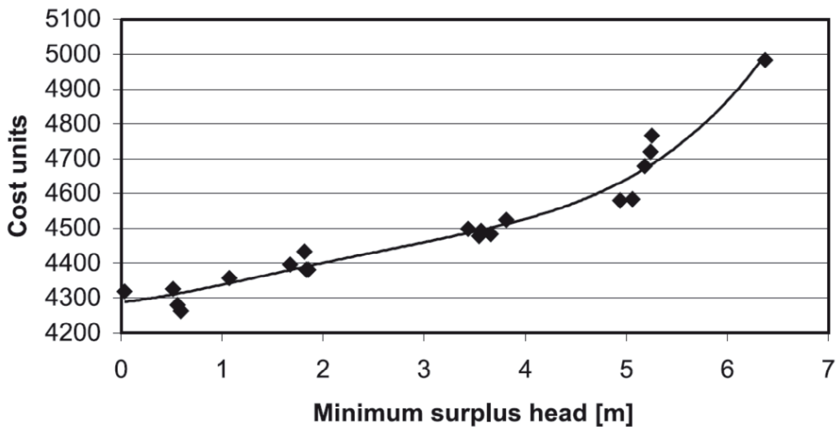

Figure 1.13 The limiting curve of the Pareto set of solutions in the cost-minimum surplus head space [Adapted from Todini (2000)]

Modified resilience index

Jayaram and Srinivasan (2008) proposed a new multi-objective formulation for the optimal design and rehabilitation of WDN, with minimization of life cycle cost and maximization of performance as objectives. Jayaram and Srinivasan (2008) reviewed several studies of WDN optimization and rehabilitation. One of their objectives was to apply Todini's (2000) resilience index I_r, among other measures, as a performance indicator of a system. However, they discovered some drawbacks if I_r index was applied for the WDN with multiple sources. According to Jayaram and Srinivasan (2008), a network with large surplus power at the demand nodes may also have a large input power value and thereby a low resilience value. Hence, Jayaram and Srinivasan (2008) rectified this drawback by introducing a modified resilience index (MI_r) the value of which is directly proportional to the total surplus power at the demand nodes as a percentage of the sum of the minimum required power at the demand nodes:

$$MI_r = \frac{\sum_{j=1}^{nn} Q_j^{req}\left(H_j - H_{\min,j}\right)}{\sum_{j=1}^{nn} Q_j^{req} H_{\min,j}} \tag{1.12}$$

Network resilience index

Based on the concept of resilience, Prasad and Park (2004) introduced a new resilience measure called network resilience. In the calculation of network resilience, the effects of both surplus power and reliable loops are considered. Following from the concept of Todini (2000), Prasad and Park (2004) defined the network resilience index as follows:

$$NI_r = \frac{\sum_{j=1}^{nn} F_j Q_j^{req}\left(H_j - H_{\min,j}\right)}{\sum_{r=1}^{nr} Q_r H_r - \sum_{j=1}^{nn} Q_j^{req} H_{\min,j}} \tag{1.13}$$

where F_j defines variation of different pipe diameters that are connected to the node j. The value of F=1 if the pipe connected to a node has the same diameter; and F<1 if pipes connected to a node have different pipe diameters. For nodes connected with only one pipe, the value F is taken to be one (Prasad and Park, 2004).

1.9.3. Resilience applications

Farmani et al. (2005) used a well-known "Anytown" network (Walski et al., 1987) as an example in order to find optimum system resilience. The resilience index I_r by Todini (2000) was considered as an objective function in the optimum design and operation of "Anytown" WDN to improve the level of reliability. Though the design of WDS under multiple loading conditions will automatically introduce a level of robustness to a network, by integrating I_r into the optimisation process will further improve the WDS reliability (Farmani et al., 2005).

Reca et al. (2008) evaluated and tested the performance of several multi-objective metaheuristics (MOMHs) to optimize the design of looped WDNs considering the Todini's resilience index I_r as a surrogate estimation of the reliability of the system. The tests at a small benchmark network (Hanoi) and a larger irrigation network (Balerma) showed that accuracy in system reliability cannot be achieved by the resilience index I_r because high I_r values do not ensure high reliability (Reca et al., 2008).

In (Wu et al. 2010) the tradeoffs between the cost and capacity reliability of a WDS were investigated based on a multi-objective genetic algorithm formulation. Wu et al. (2011) compared the surplus power factor with three commonly used network resilience measures: the resilience index (Todini, 2000), the minimum surplus head (Gessler and Walski, 1985), and the modified resilience index (Jayaram and Srinivasan, 2008). Three case studies were used to assess the suitability of the surplus power factor as a network resilience measure, and in the fourth case a Water Transmission System (WTS) with three storage tanks was studied. The results of the analysis indicate that the surplus power factor is in high correlation with the other three resilience measures investigated, and it can be used as an indicator of network resilience of a WDS (Wu et al., 2011).

Baños et al. (2011) evaluated the performance of Strength Pareto Evolutionary Algorithm 2 (SPEA2), using three resilience indexes that were described above (i.e. I_r, MI_r, NI_r). Baños et al. (2011) conducted case studies on two different WDNs and suggested that resilience indexes should consider the topology of the network in order to determine its critical points where the pressure is lower than that required.

1.10. Summary

With regard to a reliable source for drinking water for human consumption and industrial needs to be ensured, the aspects of WDN reliability have attracted notable attention. Studies have targeted to hydraulic and water quality issues as well as combining both, since the water quality is tightly connected with hydraulic aspects in WDNs (i.e., water velocity). In further studies on hydraulic reliability resilience indexes were proposed by several authors in order to assess the flexibility of WDNs against sudden failures.

Economic aspects have been of essential interest when leaping energy prices endanger everybody's welfare or water utilities raise the cost for pumping. However, joint studies of environmental aspects (carbon emissions) and pumping costs are a recent development. Until lately, many papers have mainly reported how to utilize the pumping scheduling in order to reduce the costs while accounting cheaper tariff periods for pumping. Recent studies by Cabrera et al. (2010) have analyzed all ingoing and consumed energy components of WDSs and created the concept of energy audit of WDSs.

Centuries ago the flowing water was seen as an energy producing agent (hydropower). Developments in WDSs with the requirement for pumping turned the flowing water more of an energy consuming agent. Although the water-energy nexus is widely known today, research concerning the hydraulic power transmission through the piping system is scarce. Some studies, however, have focused on the hydraulic power (the combined dimension of flow rate and pressure) and used it as one performance measure for WDSs (Park, 1998).

The development of a rehabilitation program in the late 1990s revealed a peculiarity of Tallinn WDS that was designed and built according to Soviet standards - it was heavily over-dimensioned. The consumption decreased due to industrial collapse and water tariffs increased dramatically. Due to the aging WDN the pipelines needed reconstruction and this led to the development of a rehabilitation strategy. In the beginning, the reconstruction of the WDS was mainly based on a water quality aspect - the main idea was to increase the water velocity in the system in order to reduce the water age.

By the time Tallinn WDN hydraulic models were completed, it was recognized that other aspects of network behaviour, for example the potential of hydraulic capacity of the WDSs, may not be neglected. The literature review revealed that no performance indicators are available to measure the resilience of a hydraulic system, subject to failure conditions simultaneously on the basis of both flow and pressure. This resulted in an idea to develop the surplus power factor (s) that enables determination of the reserve of hydraulic power in the WDS. Furthermore, the development of the s factor revealed several possible ways to determine it, based on how the water resistance coefficient for a WDN is calculated. The study of the hydraulic power capacity of WDSs is presented in this thesis.

2. HYDRAULIC POWER ANALYSIS OF WATER DISTRIBUTION NETWORKS

2.1. Background

Increase of the operational efficiency of water distribution networks (WDNs) has been studied throughout the years. A case study was conducted in the Tallinn WDN in order to illustrate the hydraulic capacity for existing water systems developed during past 50 years.

Resulting from the structural changes in the society, the water consumption regime has changed essentially after independence was restored in Estonia in 1991. Decrease in water consumption has led to oversized water networks in all towns. For example, in Tallinn overall water consumption has decreased over two times during the first five years of independence. Main reasons can be summarized as follows: leakage reduction, influence of the increased water price on the control of consumption (practically all consumers installed a water meter in private houses, apartments, etc.), and reduced consumption by the industry. WDNs were constructed according to the Soviet Standard SNiP for water consumption rates then that had severe influence on water quality in the network. Flow velocities were very low, the calculated retention time of water in the network before consumption was high. Therefore, extensive rehabilitation programs for WDNs were applied from 1996. The main criterion for WDN reconstruction was hydraulic reliability: to guarantee that good quality water is delivered to consumers at any time under sufficient pressure. Water age was used to forecast changes in water quality and was considered in the development of the rehabilitation strategy of WDNs.

2.2. Hydraulic Energy Transmission in a Pipe

The performance of a WDN depends on the ability to meet the demand for the volume and the pressure of the flow. To evaluate the network simultaneously on the basis of both flow and pressure, the concepts of hydraulic power capacity by the flow can be used. Hydraulic power capacity is a measure that the probability of the WDN capacity meets pressure and flow demands. It is a measure of the reliability of a system defined as a probability of a feasible flow of hydraulic power existing in the pipe or in the network.

In Vaabel et al. (2006) hydraulic energy transmission in pipeline systems was connected with system reliability and is presented as follows.

Let Q_0 and Q_1 be the flows at the inlet and at the outlet, H_0 and H_1 the heads at the inlet and at the outlet of the pipe, h the head loss due to pipe friction, and q the flow in the pipe (Figure 2.1).

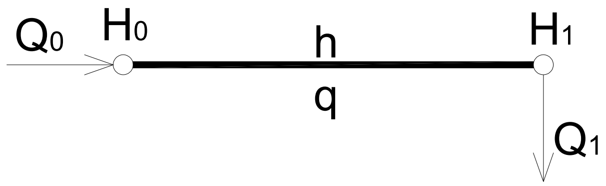

Figure 2.1 Pipe with flows, heads and head loss

We have for heads

$$H_0 = h + H_1. \tag{2.1}$$

In general, head loss can be expressed as

$$h = cq^a \tag{2.2}$$

where c is the resistance coefficient of the pipe and a is the flow exponent.
For an individual pipe

$$Q_1 = Q_0,\ q = Q_0,\ h = cQ_0^a. \tag{2.3}$$

The hydraulic power P in a pipe is defined as

$$P = \gamma QH \tag{2.4}$$

where γ is the specific weight of water, Q is the volume of the flow, and H is the head.

Respectively, we obtain

$$P_0 = \gamma Q_0 H_0 , \;\; P_d = \gamma c Q_0^{a+1} , \; P_u = \gamma Q_0 H_1 \tag{2.5}$$

where P_0 is the hydraulic power at the inlet of the pipe; P_d is the hydraulic power dissipated in the pipe, and P_u is the useful power at the outlet of the pipe.
We can write:

$$P_u = P_0 - P_d \tag{2.6}$$

or

$$P_u = \gamma\left(Q_0 H_0 - c Q_0^{a+1}\right) \tag{2.7}$$

Now let us assume that the head at the inlet of the pipe H_0 is given but the flow Q_0 can be varied. Therefore, the head H_1 depends on Q_0, i.e., $H_1(Q_0)$. Our aim is to determine the flow Q_0 such that the useful power P_u at the pipe outlet will have its maximum value.

From Eq. (2.7) under the condition $\dfrac{dP_u}{dQ_0} = 0$ it follows

$$H_0 - (a+1) c Q_{0\max}^{a} = 0 \tag{2.8}$$

or

$$Q_{0\max} = \left(\frac{H_0}{(a+1)c} \right)^{\frac{1}{a}} . \tag{2.9}$$

Now, from Eqs. (2.7) and (2.8) we obtain

$$P_{u\max} = \gamma c a Q_{0\max}^{a+1} \tag{2.10}$$

or from Eqs. (2.7) and (2.9)

$$P_{u\max} = \frac{\gamma a}{c^{1/a}} \left[\frac{H_0}{a+1} \right]^{\frac{a+1}{a}} . \tag{2.11}$$

Usually, the actual flow in the pipe is different from $Q_{0\max}$. Let us consider its effect upon the hydraulic energy transportation process in the pipe.

We define the coefficient of the critical outlet power k in the form

$$k = \frac{P_u}{P_{u\max}}. \tag{2.12}$$

Using Eqs. (2.7), (2.8) and (2.10) the coefficient k through the flows can be expressed as

$$k = \frac{a+1}{a}\left[1 - \frac{1}{a+1}\frac{Q_0^a}{Q_{0\max}^a}\right]\frac{Q_0}{Q_{0\max}}. \tag{2.13}$$

The most applicable values for the flow exponent are a =1.85 (Hazen-Williams formula) and a =2 (Darcy-Weisbach and Chezy-Manning formulas).

In Figure 2.2the coefficient of the critical outlet power k is presented as the function of the ratio $Q_0/Q_{0\max}$ if $a = 2$.

Note that if $\frac{Q_0}{Q_{0\max}} = \sqrt{3}$, we have $h = H_0$ and $H_1 = 0$.

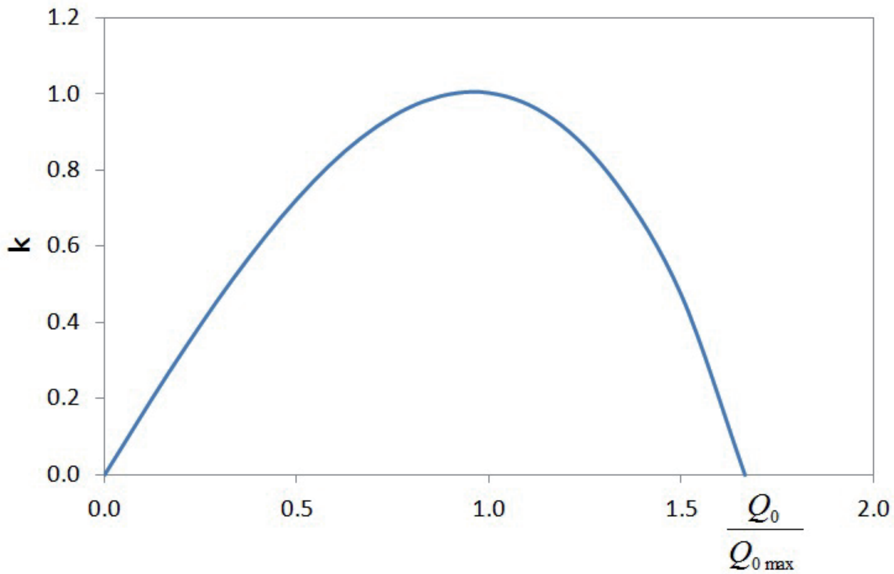

Figure 2.2 Coefficient of the critical outlet power k as the function of the ratio Q_0/Q_{0max}

The value of the coefficient k characterizes how the potentiality of hydraulic power is used by the hydraulic system. At the same time it enables us to determine the reserve of hydraulic power. For the latter, let us define the surplus power factor in the form (Figure 2.3)

$$s = 1 - k \tag{2.14}$$

The factor s characterizes the reliability of the hydraulic system. The value will vary between 1 and 0. If $s = 0$, the hydraulic system works at a maximum capacity. The increase of the value of the factor s will indicate the improvement of system reliability until it reaches its desirable value.
The considerations given here for an individual pipe can be generalized for any water distribution network.

The surplus power factor has one advantage over existing network resilience measures. The calculation of the surplus power factor does not require the value of the output pressure head of the network, it can be used to evaluate the network resilience of WTSs, whereas most of the existing surplus power-based WDS hydraulic reliability measures cannot be applied (Wu et al., 2011).

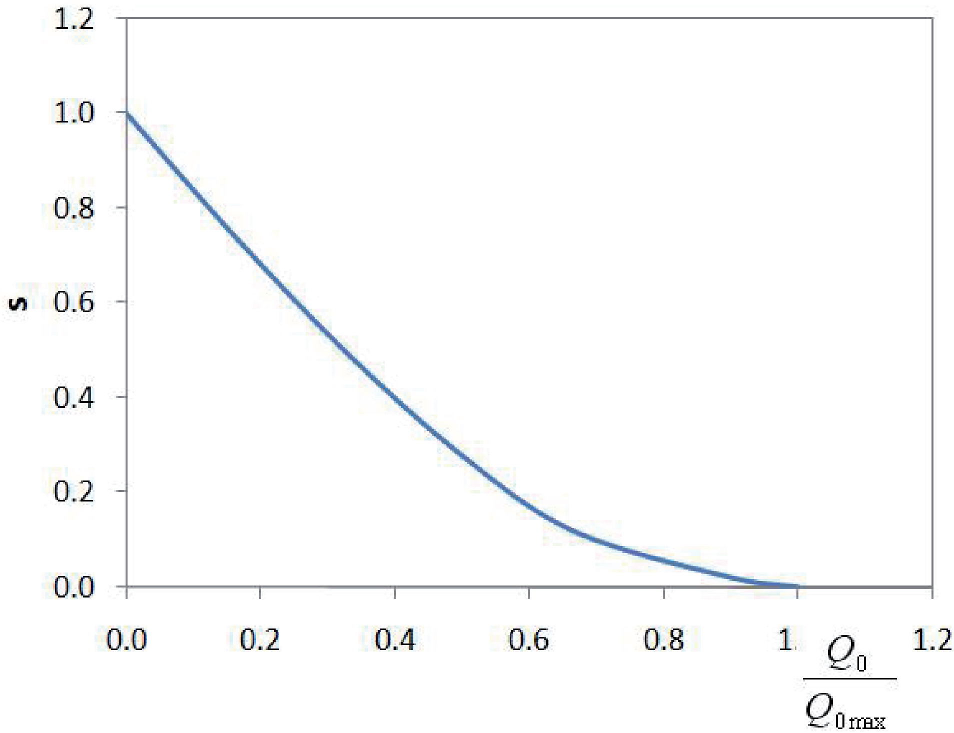

Figure 2.3 Comparison of the surplus power factor to the flow

Theoretically Q_0 could be greater than Q_{0max}, but it is not practicable. Therefore, it could be ignored in the network resilience estimation of a WDS (Wu et al. 2011).

2.3. Characteristics of a Pipe

The equation of pipe characteristic H_0=F(Q_0) is given by Eqs. (2.1) and (2.2) as

$$H_0 = cQ_0^a + H_1. \tag{2.15}$$

Equation (2.15) contains two parameters: c and H_1. Respective assemblage of characteristic curves is represented in Figure 2.4.

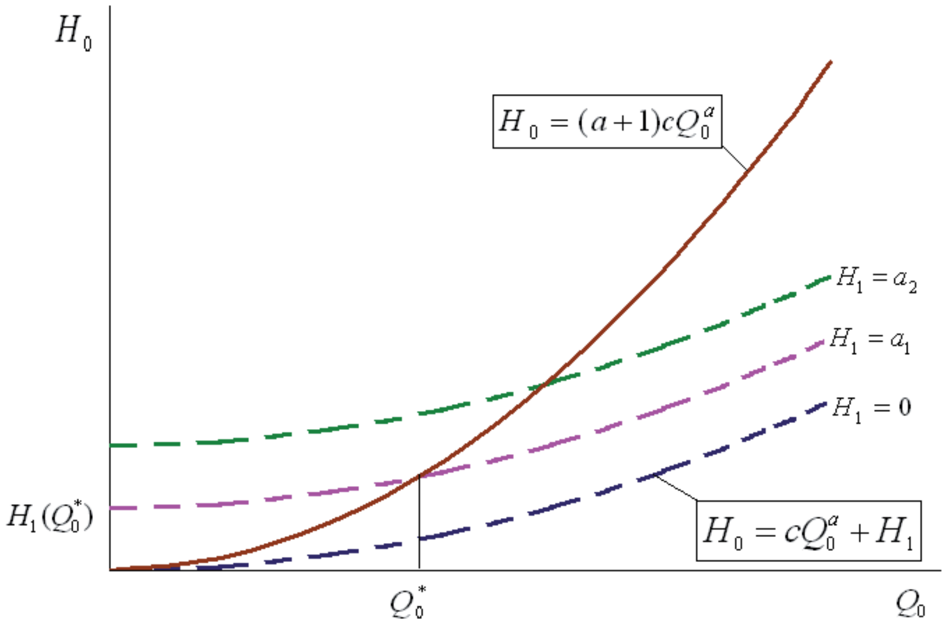

Figure 2.4 Characteristic curves of a pipe

By the determination of the flows that give the maximum power value we used the flows which depend on the head $H_1(Q_0)$. In this case Eq. (2.15) takes the form

$$H_0 = cQ_0^a + H_1(Q_0). \tag{2.16}$$

From Eq. (2.7) it follows that here

$$H_1(Q_0) = caQ_0^a . \tag{2.17}$$

The respective curve is also presented in Figure 2.4.

The intersection points of the curve (2.15) with curve (2.14) provide the values of Q_0 and $H_1(Q_0)$ by which the coefficient of the critical outlet power is maximum on the curve (2.15), i.e., k=1

2.4. Hydraulic Efficiency of the Water Distribution Network

In Vaabel et al. (2007) the hydraulic power transmission in a pipe was further developed and it was examined in a WDN as follows.

Consider a water network defined by one fixed head node (inlet) and n unknown head nodes (outlets). Let the network have p pipes and l loops. For any such network, the following identity will hold:

$$p = n + l\,. \tag{2.18}$$

Assume that the network topology is given by the following incidence matrices. The unknown head node incidence *(p* x *n)* matrix is

$$A = \left[a_{ij} \right] \text{ where } a_{ij} = \begin{cases} 1 & \text{if the flow of pipe } i \text{ enters node } j, \\ 0 & \text{if pipe } i \text{ is not connected with node } j, \\ -1 & \text{if the flow of pipe } i \text{ leaves node } j. \end{cases} \tag{2.19}$$

The direction of the flow in any pipe is of course a guess. If our prediction is wrong, the solution algorithms will give us a negative flow value.

The fixed head node incidence *(p* x *1)* matrix is

$$B = \left[b_i \right] \text{ where}$$

$$b_i = \begin{cases} 1 & \text{if the flow of pipe } i \text{ comes from the fixed head node,} \\ 0 & \text{if pipe } i \text{ is not connected with the fixed head node.} \end{cases} \tag{2.20}$$

Let the assigned nodal demands be given by *(n* x 1*)* vector Q, the unknown pipe flows defined by *(p* x 1*)* vector q and the unknown nodal heads defined by (n x 1) vector H

$$Q = \begin{pmatrix} Q_1 \\ Q_2 \\ \dots \\ Q_n \end{pmatrix}, q = \begin{pmatrix} q_1 \\ q_2 \\ \dots \\ q_p \end{pmatrix}, H = \begin{pmatrix} H_1 \\ H_2 \\ \dots \\ H_n \end{pmatrix}. \tag{2.21}$$

The assigned flow and nodal head at the inlet node are defined by Q_0 and by H_0, respectively.

The head loss *(p* x 1*)* vector h can be expressed as

$$h = Dq \tag{2.22}$$

where D is the hydraulic impedance matrix in the form

$$D = \begin{bmatrix} c_1|q_1|^{a-1} & 0 & \dots & 0 \\ 0 & c_2|q_2|^{a-1} & \dots & 0 \\ \dots & \dots & \dots & \dots \\ 0 & \dots & 0 & c_p|q_p|^{a-1} \end{bmatrix}. \tag{2.23}$$

The unknown head H_i and the flow vector components q_k are determined from the following energy and mass conservation laws:

$$AH + Dq = -BH_0,\ A^T q = Q,\ -B^T q = Q_0 \tag{2.24}$$

where A^T and B^T are the transpose of matrices A and B. Let us note that in Eq. (2.24) the first expression is a system of p nonlinear equations, the second is a system of n linear equations and the last one has a dimension of (1 x 1) – equation.

Here the continuity of the flow rate

$$Q_0 = \sum_{i=1}^{n} Q_i \tag{2.25}$$

holds.

The network characteristic is expressed as

$$H_0 = F(Q_0) \tag{2.26}$$

where the function F is determined through the system of equations (2.24) – (2.25). Many software packages are available to solve the system of the WDN equations and to find the function F.

Let us now determine the flows that maximize the outlet hydraulic power P_u. We take

$$Q = Q_0 \widetilde{Q},\ \ q = Q_0 \widetilde{q},\ H = Q_0^a \widetilde{H},\ D = Q_0^{a-1} \widetilde{D} \tag{2.27}$$

where $\widetilde{Q}$, $\widetilde{q}$, $\widetilde{H}$ and $\widetilde{D}$ = variable correction factors for tentative values of nodal demands, flows, heads and the hydraulic impedance matrix. This approach allows us to consider the influence of diurnal changes of water consumption in the network. Likewise, the time variability of water demand is taken into consideration.

Then Eqs. (2.24) – (2.25) can be written in the form

$$\left(A\widetilde{H} + \widetilde{D}\widetilde{q}\right) Q_0^a = -BH_0 \tag{2.28}$$

and

$$A^T \widetilde{q} = \widetilde{Q},\ -B^T \widetilde{q} = 1,\ \sum_{i=1}^{n} \widetilde{Q}_i = 1. \tag{2.29}$$

By multiplying both sides with B^T and taking into consideration that $B^T B = \beta$, we obtain from Eq. (2.28)

$$H_0 = -\frac{1}{\beta} B^T \left(A\widetilde{H} + \widetilde{D}\widetilde{q} \right) Q_0^a \tag{2.30}$$

where β is the number of units among the matrix B elements. Let us note that the dimension of (2.30) is (1 x 1).

With the denotation

$$C_{tot} = -\frac{1}{\beta} B^T \left(A\widetilde{H} + \widetilde{D}\widetilde{q} \right) \tag{2.31}$$

Eq. (2.30) takes the form

$$H_0 = C_{tot} Q_0^a \, . \tag{2.32}$$

Here C_{tot} is the resistance coefficient of the water network that characterizes the condition when all hydraulic power has been dissipated in the system. In general, it depends on $\widetilde{q}$ and $\widetilde{D}$ elements and on the head in one point of the network. Eq. (2.32) characterizes the condition when head loss in the system equals head at the input. In practice, WDSs operate in the condition when the input head is greater than the head loss through the piping system, as described in Eq. (2.1). This provides necessary pressure in consumer nodes. From Eq. (2.31) the dimension of C is (1 x 1). The values of $\widetilde{H}$, $\widetilde{D}$ and $\widetilde{Q}$ are changing in time, but the value of C from Eq. (2.32) is constant.

The hydraulic power of the water network can be expressed as

$$P_0 = \gamma Q_0 H_0 \, , P_d = \gamma (h)^T q, \; P_u = P_o - P_d \text{ or} \tag{2.33}$$

$$P_0 = \gamma Q_0 H_0 , \; P_d = \gamma (Dq)^T q, \; P_u = \gamma \left[Q_0 H_0 - (Dq)^T q \right] \text{ and} \tag{2.34}$$

$$(Dq)^T q = (\widetilde{D} Q_0^{a-1} Q_0 \widetilde{q})^T Q_0 \widetilde{q} = Q_0^{a+1} \widetilde{q}^T \widetilde{D} \widetilde{q} \tag{2.35}$$

where P_0 is the hydraulic power at the inlet of the network; P_d is the hydraulic power dissipated in the pipes, and P_u is the useful power at the outlets of the network.

From Eqs. (2.22), (2.33) and (2.34) it follows that

$$C = \tilde{q}^T \tilde{D} \tilde{q}, \tag{2.36}$$

where C is resistance coefficient of the water distribution network.

In practice, network equations can be solved with appropriate hydraulic software, for example EPANET (Rossmann, 2000).

For a=2 and based on Eq. (2.36), C can also be expressed as

$$C = \frac{\sum_i (h_i \cdot q_i)}{Q_0^3}, \tag{2.37}$$

where h_i is head loss in pipe i, and q_i is flow in pipe i.

If the results for head losses and flows in all pipes in the system are determined by the hydraulic solver, h_i and q_i values could be exported to the spreadsheet application and C value determined with Eq. (2.37).

Defining the coefficient of the hydraulic efficiency of the network η_n in the form

$$\eta_n = \frac{P_u}{P_0}, \tag{2.38}$$

we have

$$P_u = \eta_n P_0 \text{ or } P_u = \gamma \eta_n H_0 Q_0. \tag{2.39}$$

From Eqs. (2.33) and (2.38) we obtain

$$\eta_n = 1 - \frac{C Q_0^a}{H_0}. \tag{2.40}$$

Now we assume that the head of the inlet of the network H_0 is given but the flow Q_0 can be varied. Let us determine this flow such that the useful power P_u will have the maximum value.

From Eq. (2.39) under the necessary extremum condition

$$\frac{dP_u}{dQ_0} = 0 \tag{2.41}$$

it follows that

$$\frac{d\eta_n}{dQ_0} Q_0 + \eta_n = 0 \tag{2.42}$$

or

$$Q_0 \frac{d}{dQ_0} \ln \eta_n = -1 \quad . \tag{2.43}$$

Substituting Eq. (2.40) into Eq. (2.42) we obtain

$$Q_{0\,\max} = \left[\frac{H_0}{(1+a)C} \right]^{\frac{1}{a}} \tag{2.44}$$

Eq. (2.44) for the network coincides with Eq. (2.9) for an individual pipe. Therefore, Eqs. (2.10) to (2.13) and (2.14) respectively, are applied also to the water distribution network as a whole.

Now from Eqs. (2.40) and (2.44) it follows

$$\eta_n = \frac{a}{1+a} \; . \tag{2.45}$$

So for any H_0 the hydraulic efficiency of the network η_n at the maximum value of P_0 is constant, given by Eq. (2.45) (Figure 2.5).

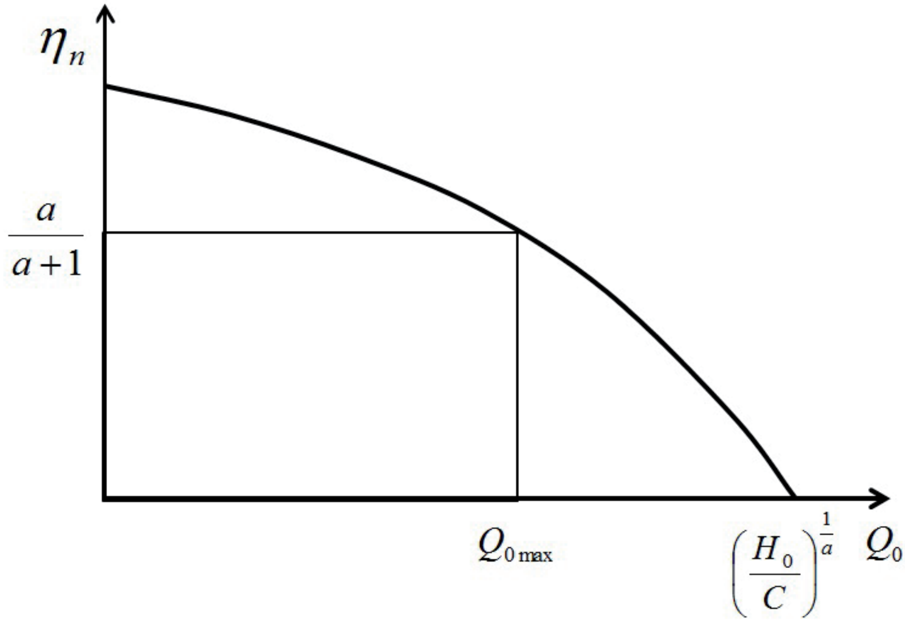

Figure 2.5 Hydraulic efficiency of the network as the function of Q_0

2.5. Energetic Efficiency of Water Supply System by Constant Head at Inflow

Let us consider now the water distribution network and the pumping station in joint operation.

Let P be the adsorbed power by operating the pumping station with the network. Then we can write

$$P = \frac{P_0}{\eta} \,, \tag{2.46}$$

where η is the general efficiency of the pumping station.

From Eqs. (2.39) and (2.46) we obtain

$$P_u = \eta_n \eta P \,. \tag{2.47}$$

Denote by η_s the efficiency of water distribution system, i.e.

$$\eta_s = \eta_n \eta . \tag{2.48}$$

Assume that η can be approximated in the form

$$\eta(Q,H) = \alpha_0 + \alpha_1 Q + \alpha_2 H + \alpha_3 Q^2 + \alpha_4 H^2 + \alpha_5 QH. \tag{2.49}$$

Determine now the maximum value of the efficiency of the distribution system by a constant H_0.

From the condition

$$\frac{d\eta_s}{dQ_0} = 0 \tag{2.50}$$

for the constant H_0, Eq. (2.49) takes the form

$$\eta(Q_0, H_0) = b_0 + b_1 Q_0 + b_2 Q_0^2 , \tag{2.51}$$

where

$$\begin{aligned} b_0 &= \alpha_0 + \alpha_2 H_0 + \alpha_4 H_0^2 , \\ b_1 &= \alpha_1 + \alpha_5 H_0 , \\ b_2 &= \alpha_3 \end{aligned} \tag{2.52}$$

From Eqs. (2.40), (2.49), (2.50) and (2.51) we have

$$c_1 Q_0^{a+1} + c_2 Q_0^{a} + c_3 Q_0^{a-1} + c_4 Q_0 + c_5 = 0 , \tag{2.53}$$

where

$$
\begin{aligned}
c_1 &= (a+2)b_2 C, \\
c_2 &= (a+1)b_1 C, \\
c_3 &= ab_0 C, \\
c_4 &= -2b_2 H_0, \\
c_5 &= -b_1 H_0.
\end{aligned} \tag{2.54}
$$

Characteristic curves for networks can be graphically described similarly to those of a single pipe (Figure 2.4).

2.6. Energetic Efficiency of a Water Supply System by a Given Characteristic Curve of a Pump

In Koppel et al. (2009) the pumping efficiency of a WDS was examined as follows.

Let us consider the water distribution network and the pumping station in joint operation. We assume that the head by the determination of maximum useful power is not constant but it changes in accordance with the head-discharge relationship of a given pump. Useful power of the WDN can be expressed as

$$P_u = \gamma\left(Q_0 H_0 - C Q_0^{a+1}\right) \tag{2.55}$$

Assume that the head characteristic curve is described by the function $H=H(Q)$. Substituting this function into Eq. (2.55), instead of H_0, we have

$$P_u = \gamma\left[Q_0 H(Q_0) - C Q_0^{a+1}\right]. \tag{2.56}$$

In this case

$$\eta_n = 1 - \frac{C Q_0^a}{H(Q_0)}. \tag{2.57}$$

Now from Eq. (2.41) we obtain

$$H(Q_0) + Q_0 \frac{dH}{dQ_0} - (a+1)CQ_0^a = 0 \qquad (2.58)$$

Let the head characteristic curve be a power function in the form

$$H = H_c - gQ_0^m, \qquad (2.59)$$

where H_c is a pump head at zero flow, g and m coefficients which determine the pump curve shape.

Substituting Eq. (2.59) into Eq. (2.58) we obtain

$$H_c - (m+1)gQ_0^m - (a+1)CQ_0^a = 0. \qquad (2.60)$$

Solution of this equation – Q_{0max} gives us the inflow to the network which maximizes the useful power P_u.

Let us consider a special case. Assume that a=2 and g<<C and let m be a natural number.

Take

$$Q_{0\max} = Q_{0(0)} + gQ_{0(1)} + g^2 Q_{0(2)} + \ldots \qquad (2.61)$$

Substituting Eq. (2.61) into Eq. (2.60), we have

$$\begin{aligned} &H_c - (m+1)g(Q_{0(0)}^m + mgQ_{0(0)}^{m-1}Q_{0(1)} + ..) - \\ &-3C\left[Q_{0(0)}^2 + 2gQ_{0(0)}Q_{0(1)} + g^2\left(Q_{0(1)}^2\right) + 2g^2 Q_{0(0)}Q_{0(2)} + \ldots\right] = 0. \end{aligned} \qquad (2.62)$$

From here for two first terms in series (2.61) we obtain

$$\begin{aligned} &H_c - 3CQ_{0(0)}^2 = 0, \\ &(m+1)Q_{0(0)}^m + 6CQ_{0(0)}Q_{0(1)} = 0, \end{aligned} \qquad (2.63)$$

Therefore

$$Q_{0\max} = \sqrt{\frac{H_C}{3C}} - \frac{1+m}{6C} g \left(\sqrt{\frac{H_C}{3C}} \right)^{m-1} . \tag{2.64}$$

From Eq. (2.64) it follows for m=2

$$Q_{0\max} = \left(1 - \frac{1}{2} \frac{g}{C} \right) \sqrt{\frac{H_C}{3C}} \quad , \tag{2.65}$$

and for m=3

$$Q_{0\max} = \sqrt{\frac{H_C}{3C}} - \frac{2}{9} \frac{g}{C} H_C . \tag{2.66}$$

Let us now determine the flow which gives the maximal value of the efficiency η_n.

By taking the two first terms from an appropriate power series, we obtain from Eqs. (2.40) and (2.59)

$$\eta_n(Q_0) = 1 - \frac{CQ_0^a}{H_C - gQ_0^m} \quad . \tag{2.67}$$

Assume that $gQ_o^m << H_c$, then Eq. (2.66) can be written as

$$\eta_n(Q_0) = 1 - \frac{CQ_0^a}{H_C} - \frac{gC}{H_C^2} Q_0^{a+m} \quad . \tag{2.68}$$

From Eqs. (2.49) and (2.59) it follows

$$\eta[Q_0, H_0] = d_0 + d_1 Q_0 + d_2 Q_0^2 + d_m Q_0^m + d_{m+1} Q_0^{m+1} + d_{2m} Q_0^{2m} \tag{2.69}$$

where

$$
\begin{aligned}
d_o &= \alpha_0 + H_0\alpha_2 + H_c^2\alpha_4, \\
d_1 &= \alpha_1 + H_c\alpha_5, \\
d_2 &= \alpha_3, \\
d_m &= -g(\alpha_2 + 2\alpha_4), \\
d_{m+1} &= -g\alpha_5, \\
d_{2m} &= g^2\alpha_4.
\end{aligned}
\tag{2.70}
$$

Now the value of the flow Q_0 that maximizes the efficiency of the water distribution system can be determined by

$$
\frac{d\eta_n}{dQ_0}\eta + \eta_n \frac{d\eta}{dQ_0} = 0, \tag{2.71}
$$

where η_n and η are determined through Eqs. (2.68) and (2.69).

By utilizing the theory described in sections (2.5) and (2.6), it is possible to determine the energy efficiency of a WDS consisting of a WDN and a pumping station. However, due to the lack of necessary information for pumps the theory is not tested for in-service WDSs yet.

3. DEVELOPMENT OF SURPLUS POWER FACTOR ANALYSIS

3.1. Background

As mentioned in the previous chapter, an extensive rehabilitation program for Tallinn city WDNs started in 1996. At that time information about the WDS characteristics (pipes, demands, water quality, etc) was scattered and no hydraulic models about the system were available. Therefore data collecting took several years (see Figure 1.1) and initial hydraulic models were developed. In addition, several years were spent collecting adequate measurement data about actual pressures and flows in the system. In 2003 the hydraulic models were calibrated to create a basis for a further rehabilitation program.

The early rehabilitation decisions were mainly based on water quality issues. Since the relict from the Soviet era was an over-dimensioned system, the main concern was slow velocities in the WDN that caused long water age and deteriorated water quality. Therefore, the plan of action consisted of reduction of the diameters where pipes needed replacement due to bad installation quality.

Continuous rehabilitation was conducted also on pumping stations. Since practically no water towers were in operation during that time, it was reasonable to replace old pumps with more energy efficient ones and add frequency converters to be able to regulate pump speeds according to demand. In some cases when pumps performed well in terms of energy efficiency, only frequency converters were installed.

When the hydraulic models provided a good basis for analysis in terms of water quality and overall performance in the WDN, it was realised that to evaluate the WDN piping it was essential to study hydraulic power transmission. Based on the studies conducted by Park et al. (1998) the theory of hydraulic power transmission was further developed. The idea was to find optimum solutions for the hydraulic power transmission in the system and at the same time to analyse the reliability of the WDSs. Therefore, the surplus power factor was introduced.

Initially, optimum solutions were analysed in each pipe section of the WDN and the results were averaged to the whole network. Thus, although the theory could be adequately applied to each pipe section between the numerous nodes in the WDN, the results for the whole WDN were not as expected. The reason is that since the optimum power loss for the most effective power transmission is one third of the initial head and if this approach is applied to all the pipes in the WDN, the customers would end up with no water (i.e., too high power loss in the system).

Then the idea was to focus on the power loss between the source and the target node. In this case the target node was considered as the characteristic point in the

WDN, for example the most remote node in the system, the junction where future WDN extension would be carried out or just the junction with the highest demand.

Finally, the theory for one pipe was applied to the WDN using matrix equations as described in (section 2.4).

As a result, three different approaches available to calculate *s* factor for WDNs could be outlined as follows:

- **Version 1**
  s is determined individually for each pipe and the result is averaged over all pipes in the WDN. This will give an average *s* value of the system, but cannot be well utilized in practical applications, since the short pipeline sections account for small head loss and therefore high *s* values. Still, this approach could be applied for WTSs.
- **Version 2**
  s is determined for certain nodes in the system through head loss between the source and the target node. This will give several *s* values around the system according to the node position. This approach is well applicable to check the nodes in farther sections of the network whether future extension of the WDN is possible.
- **Version 3**
  s is determined through the overall network resistance coefficient which accounts the head losses in each pipe. This will give an average *s* value of the system and is probably well applicable in order to assess the overall performance of WDS.

3.2. Surplus Power Factor Calculation Algorithm, Version 1

This approach (Report 2000, Tallinn University of Technology) was based on the idea to determine optimum velocities and flows in WDN pipes considering maximum hydraulic energy transmission in a single pipe. Later, the idea was elaborated to WDNs that are the systems of multitude of pipes.

The approach can be described as follows.

If the flow is known in the pipe, head loss can be defined with the Darcy-Weisbach equation as

$$h = 8\frac{\lambda L {Q_0}^2}{\pi^2 g D^5}, \tag{3.1}$$

where λ is the friction factor of the pipe; L is the pipe length (meters); D is the pipe diameter (meters) and g is the gravitational constant.

By using Eqs. (2.1), (2.5) and (2.6) the useful power at the outlet of the pipe can be denoted as

$$P_u = \gamma Q_0 \left(H_0 - h\right). \tag{3.2}$$

Substituting Eq. (3.1) into Eq. (3.2), we obtain

$$P_u = \gamma \left(H_0 Q - 8 \frac{\lambda L}{\pi^2 g D^5} Q^3 \right). \tag{3.3}$$

From Eq. (3.3) under the condition $\dfrac{dP_u}{dQ_0} = 0$ it follows

$$H_0 = 3 \cdot 8 \frac{\lambda L}{\pi^2 g D^5} Q^2. \tag{3.4}$$

Eq. (3.4) clearly shows that optimum power transmission in the pipe equals one third of the head at the beginning of the pipe. Therefore, an optimum flow when the hydraulic power transmission is maximum can be denoted as

$$Q_{0\max} = \sqrt{\frac{\pi^2 g D^5 H_0}{24 \lambda L}}, \tag{3.5}$$

or

$$Q_{0\max} = \frac{\pi D^2}{4} \sqrt{\frac{D}{\psi}}, \tag{3.6}$$

where

$$\psi = \frac{3 \lambda L}{2 g H_0}. \tag{3.7}$$

Optimum velocity can be defined as

$$v_{op} = \sqrt{\frac{D}{\psi}}. \tag{3.8}$$

As it was recognized later and as presented in section 2.2, the latter flows and velocities can rather be defined as "critical" not "optimal". As could be seen from Figure 2.2 and Figure 2.3, the maximum potentiality of hydraulic power is used when the value k reaches one and then starts to decrease again. At the same time if the surplus power factor s reaches zero it cannot be defined as optimum since there will be no resilience left. Therefore, the term "critical" was found more appropriate.

Based on the latter formulation, critical flows and velocities in the pipes were calculated according to certain D and ψ values, as presented in Table 3.1 and Table 3.2.

Table 3.1 Critical velocity (m/s) in terms of power transmission

$D \setminus \psi$	**0.01**	**0.05**	**0.1**	**0.5**	**1.0**	**5.0**	**10.0**
0.10	3.16	1.41	1.00	0.45	0.32	0.14	0.10
0.20	4.47	2.00	1.41	0.63	0.45	0.20	0.14
0.30	5.48	2.45	1.73	0.77	0.55	0.24	0.17
0.40	6.32	2.83	2.00	0.89	0.63	0.28	0.20
0.50	7.07	3.16	2.24	1.00	0.71	0.32	0.22
0.60	7.75	3.46	2.45	1.10	0.77	0.35	0.24
0.80	8.94	4.00	2.83	1.26	0.89	0.40	0.28
1.00	10.00	4.47	3.16	1.41	1.00	0.45	0.32
1.50	12.25	5.48	3.87	1.73	1.22	0.55	0.39

Table 3.2 Critical flow (m^3/s) in terms of power transmission

$D \setminus \psi$	**0.01**	**0.05**	**0.1**	**0.5**	**1.0**	**5.0**	**10.0**
0.10	0.02	0.01	0.01	0.00	0.00	0.00	0.00
0.20	0.14	0.06	0.04	0.02	0.01	0.01	0.00
0.30	0.39	0.17	0.12	0.05	0.04	0.02	0.01
0.40	0.79	0.36	0.25	0.11	0.08	0.04	0.03
0.50	1.39	0.62	0.44	0.20	0.14	0.06	0.04
0.60	2.19	0.98	0.69	0.31	0.22	0.10	0.07
0.80	4.50	2.01	1.42	0.64	0.45	0.20	0.14
1.00	7.85	3.51	2.48	1.11	0.79	0.35	0.25
1.50	21.64	9.68	6.84	3.06	2.16	0.97	0.68

Hydraulic models were used in order to determine velocities and flows in the pipes in the Tallinn WDS. Since the velocities in the pipes were rather slow, it was concluded that the system worked very inefficiently in terms of hydraulic power transmission (Report 2000, Tallinn University of Technology). The flowchart for the calculation of the surplus power factor according to version 1 is presented in Figure 3.1.

The results were slightly misinterpreted since the analysis was done only on pipe-by-pipe basis in the WDN. This type of an approach could be applied to long WTSs or simplified network models rather than to a WDN with many nodes and short pipe sections between them.

Therefore, the development of the theory of hydraulic power transmission was more focused on WDNs, as described in section 2.4.

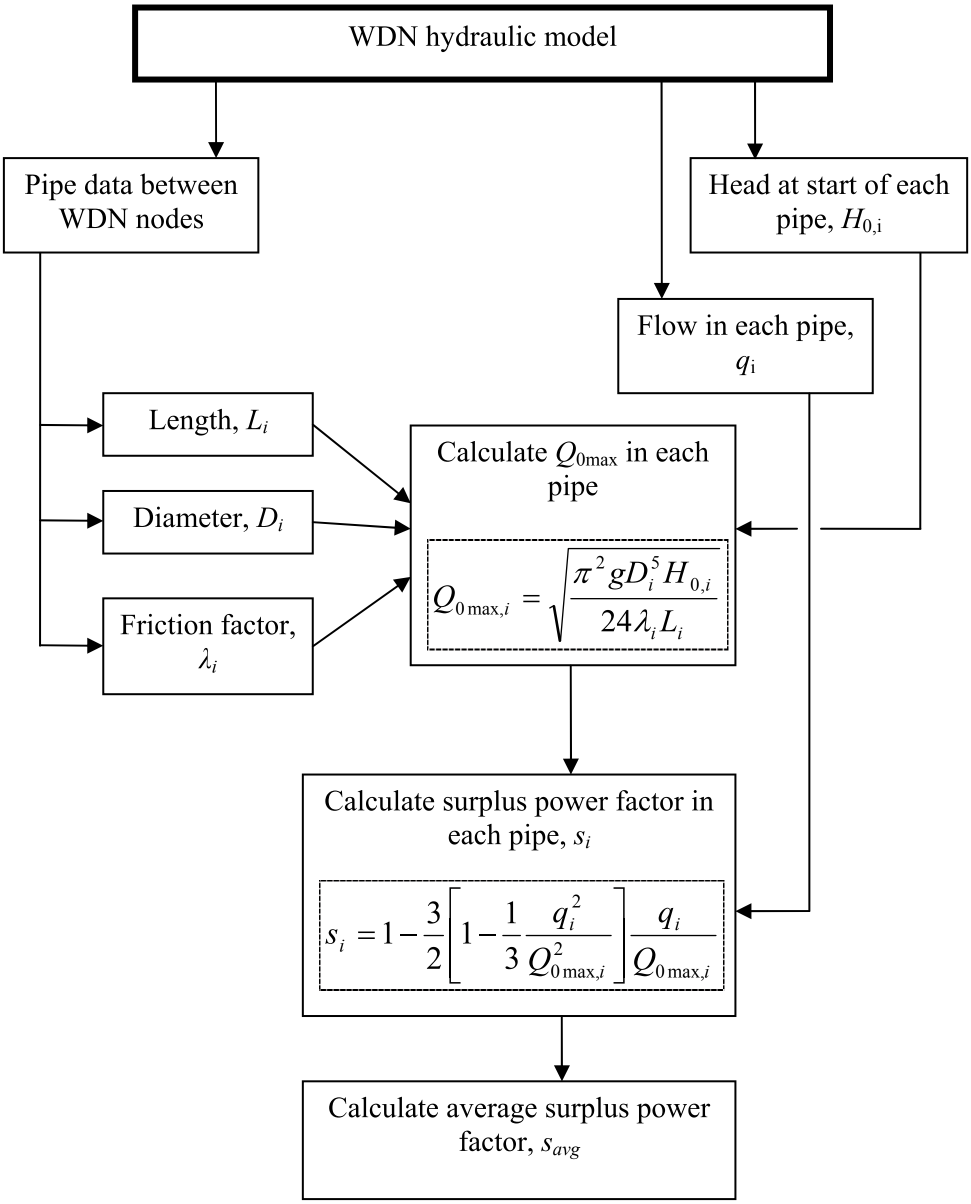

Figure 3.1 Flowchart of the surplus power factor calculation algorithm, version 1

3.3. Surplus Power Factor Calculation Algorithm, Version 2

Based on Eqs. (2.1) and (2.2), the main factor contributing to the surplus power factor calculation is the head loss between the source and the target nodes. As defined by Eq. (2.32), the same principle could be applied to WDNs.

Thus, the surplus power factor in the WDN can be only regarded from a certain node's perspective. It means the farther the node from the source node (i.e. pumping station), the greater the head loss at the target node and therefore more hydraulic power is dissipated, resulting in smaller s. This approach needs no analysis of the target and source node connection to the pipes or of the exact flow route between the nodes. For example, main lines connecting the farthest node in the system with the pumping station are presented in Figure 3.2. The flowchart for the calculation of the surplus power factor is presented in Figure 3.3.

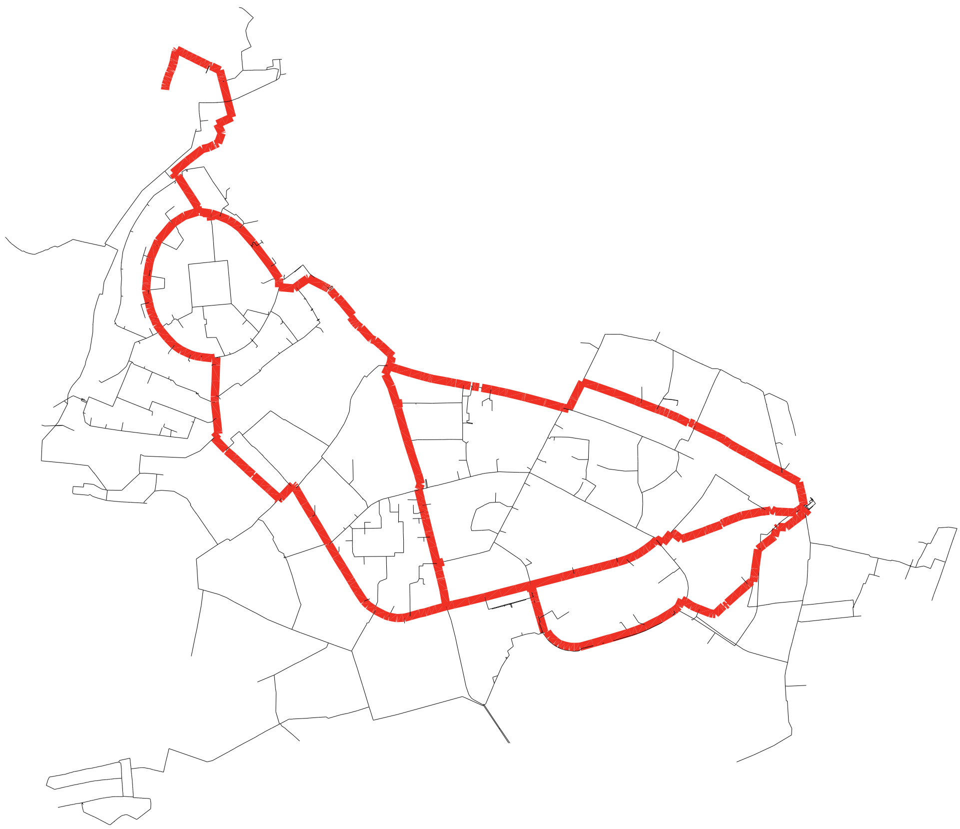

Figure 3.2 Flow paths between the source and the target node

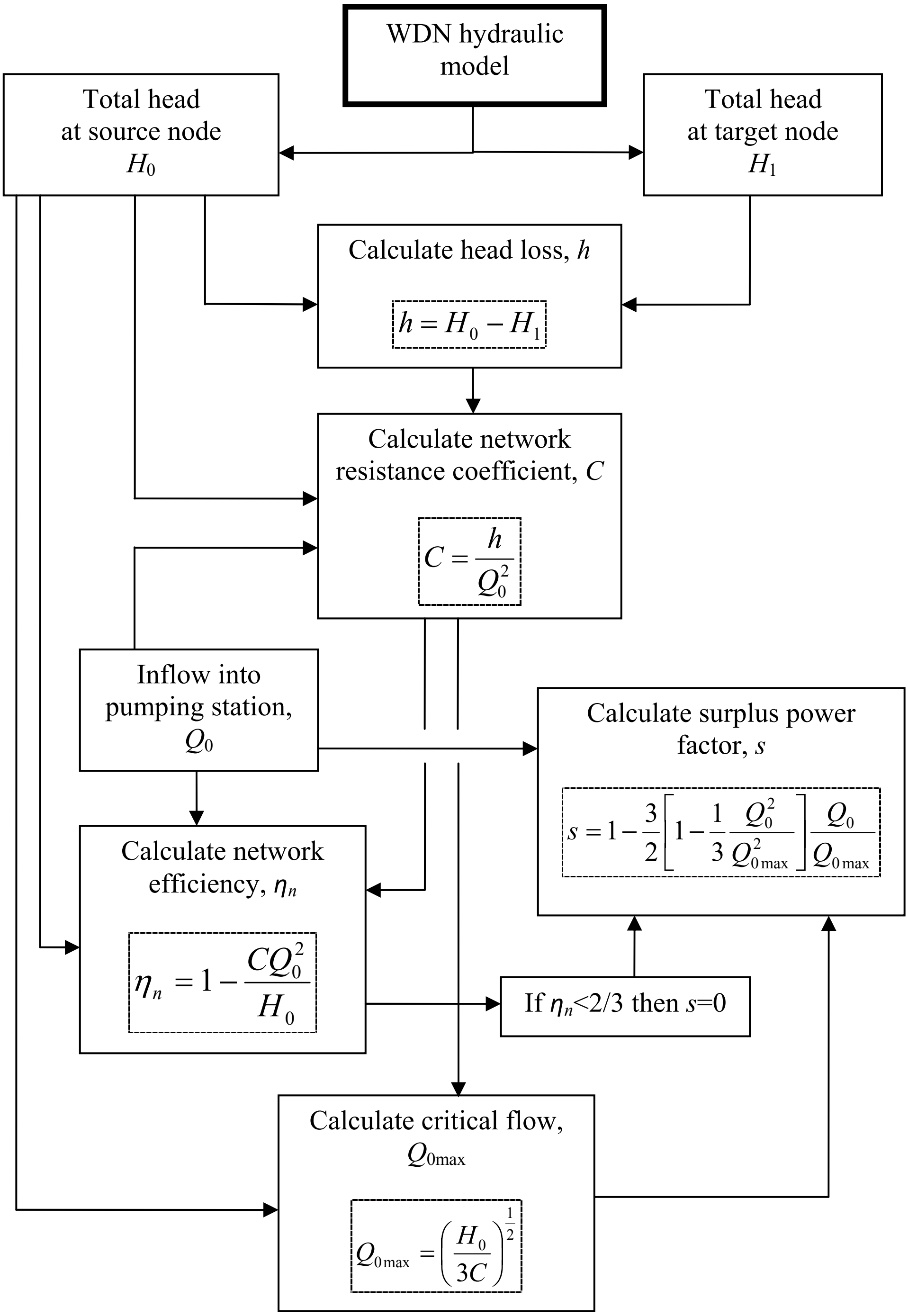

Figure 3.3 Flowchart of the calculation algorithm of the surplus power factor, version 2

3.4. Surplus Power Factor Calculation Algorithm, Version 3

The analysis of the surplus power factor revealed that an accurate s value depends on the WDN resistance coefficient C. Therefore, it was crucial to find an approach for determining the C value that will characterize the whole WDN, not just the resistance of individual pipes or the resistance between certain nodes (i.e. the source and the target nodes). Determination of the resistance coefficient C is presented in section 2.34 with Eq. (2.37). Figure 3.4 presents the flowchart of the calculation algorithm of the surplus power factor according to version 3.

Although all three versions of C calculations are valid, the one presented by Eq. (2.37) seems to characterize in-service WDNs best. In order to compare the different methods of C calculations, a case study for an in-service WDN was conducted, as presented in Chapter 4.

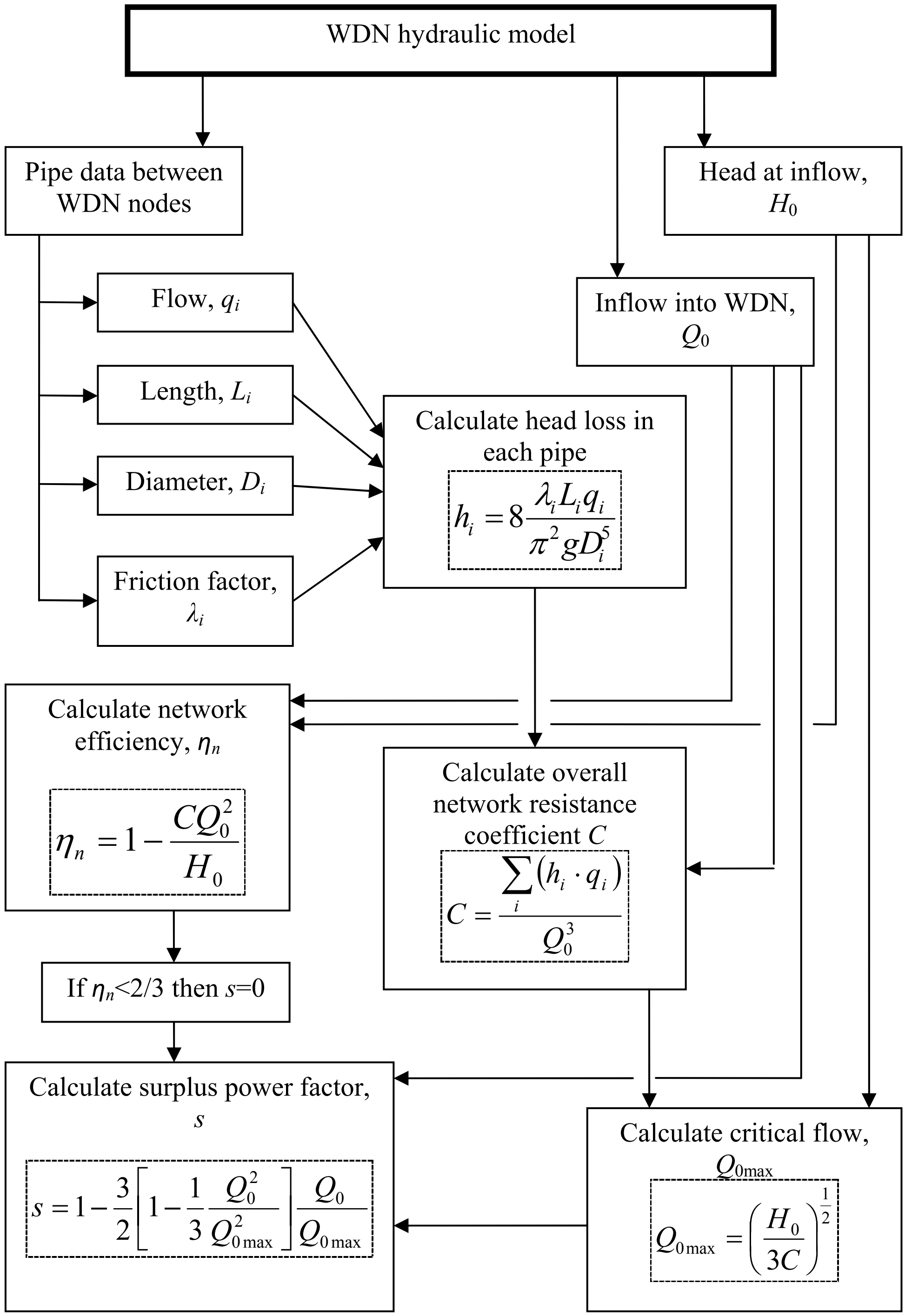

Figure 3.4 Flowchart of the calculation algorithm of the surplus power factor, version 3

4. CASE STUDIES

4.1. General

In order to illustrate surplus power factor calculations in different real-world situations two types of WDNs were used. Firstly, a case study was conducted on a regular WDS the source of which is the booster pumping station and a WDN that contains residential, industrial and commercial consumers. This type of a water distribution system has great impact on diurnal variation of demand and therefore the surplus power factor varies also substantially along with the demand variation.

The second case study was carried out on a WTS that carries water from boreholes groups into a water treatment plant. This type of a system has stable flow into the water collection reservoir and therefore it has constant surplus power factor throughout the day.

The results obtained by using one of the three approaches are compared in section 4.2.4.

4.2. Surplus Power Factor Analysis of the Õismäe-Mustamäe WDN

In order to illustrate the hydraulic power reserve, i.e., the surplus power factor s and the WDN efficiency η_n in an existing WDS, a case study was carried out with a medium-sized WDS in Tallinn.

The case study covered the Õismäe-Mustamäe area, which is one of several independent pressure zones in the Tallinn WDS. The total length of pipelines in this pressure zone is approximately 85 km. Since all consumers are equipped with flow meters, the water company has a relatively good overview about the demand and unaccounted water distribution in the system. Unaccounted water and leakages of around 15% have been measured by the water company. The main problem found in the Õismäe-Mustamäe WDN was over-dimensioned pipelines producing relatively small velocities that affect the water quality before it reaches a consumer. When the system was designed, the overall consumption was approximately two times higher than at present. The demand pattern of the water system follows mostly that of the residential one, i.e., small private houses or apartment buildings (85% of consumers). Although large consumers (some factories) are represented with their real demand patterns, their effect on the overall diurnal demand variation is negligible. Still, a large bakery in the area has some impact on the variation of the consumption peak hour. The maximum peak demand is at 6:00 am, the minimum at 2:00 am and the average demand at 4:00 pm. The distribution of pipe diameters and age are presented in Figure 4.1 and Figure 4.2.

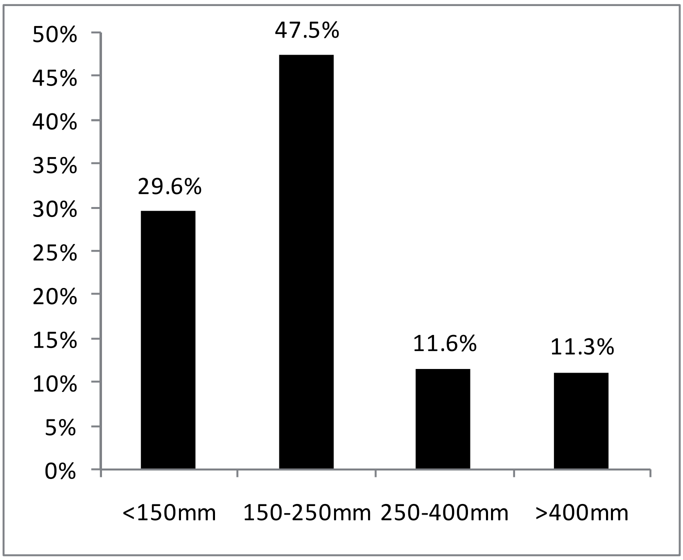

Figure 4.1 Pipe diameter distribution of the Õismäe-Mustamäe WDN

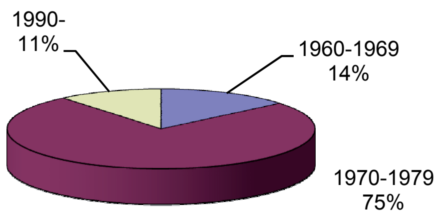

Figure 4.2 Pipe age distribution of the Õismäe-Mustamäe WDN

Demand is applied to consumer groups according to their characteristics. Demand patterns for consumer groups are presented in Appendix A. The WDN model consists of 2480 nodes and 2560 pipes.

The system is supplied via two booster pumping stations: P1 and P2 (Figure 4.3). Since the real demand is significantly lower than that designed, the pumping station P2 is put into operation only under peak demand conditions. Under normal diurnal demand conditions it is switched off. Also, when the model was calibrated, P2 was in the standby position. Since the consumption and therefore the pressure loss in the system will drop significantly during the night time, the pressure in the pumping station P1 is lowered. As Figure 4.3 shows, choice of the location of the main source (P1) for energy is not well-grounded to ensure an even distribution of power, though it is compensated by higher ground level than most of the nodes in the network.

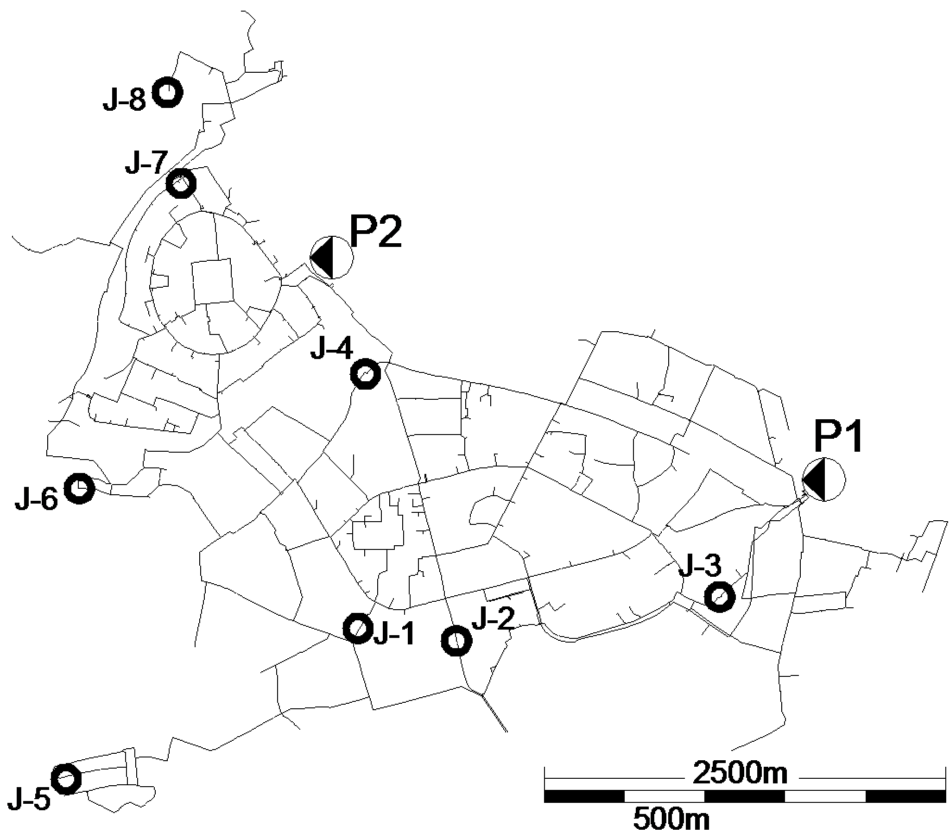

Figure 4.3 Layout of the Õismäe-Mustamäe hydraulic model with characteristic junctions.

4.2.1. Calculation for version 1

As described in section 3.2, this approach calculates the *s* factor for each pipe. After running the EPANET hydraulic model, the head loss in each pipe according to Eq. (3.1) was determined. After $Q_{0\max}$ was determined according to Eq. (3.5), the surplus power factor for each pipe was found according to Eqs. (2.13) and (2.14).

The *s* factor was calculated for daily maximum, minimum and average demand conditions as well as for maximum demand condition in fire flow situation. The fire flows were selected based on Estonian local standards (EVS 812-6:2012) that define the minimum required fire flow from the utility network based on the volume of the building. If the required fire flow for a certain building, e.g., a factory, is larger than the utility network could provide in the area, the property owner has to guarantee the fire flows from the local water tank and the pump system.

The location for the fire flow nodes is presented in Figure 4.3 and the fire flows were selected 15 L/s for each node, except for node J-5 (fire flow was selected 10 L/s).

The results for the *s* factor calculation according to version 1 are presented in Table 4.1 and Table 4.2 and respective graphs in Figure 4.4 and Figure 4.5.

Table 4.1 Results of surplus power factor calculation at different time steps

	6:00am	4:00pm	2:00am
Q_0	285.38	198.37	58.51
H_0	71.64	71.53	60.24
s_{avg}	96.0%	97.7%	98.8%
s_{min}	69.3%	81.5%	90.0%
s_{max}	99.9%	99.9%	100.0%

Table 4.2 Results of surplus power factor calculation at 6:00 am + fire flow at a given node

	J-1	J-2	J-3	J-4	J-5	J-6	J-7	J-8
Q_0	300.38	300.38	300.38	300.38	295.38	300.38	300.38	300.38
H_0	66.14	66.14	66.14	66.14	70.02	66.14	66.14	66.14
s_{avg}	95.5%	95.6%	95.7%	95.0%	95.0%	95.1%	95.2%	94.9%
s_{min}	64.2%	60.1%	67.8%	55.9%	43.5%	62.6%	62.6%	61.8%
s_{max}	100.0%	99.9%	99.9%	99.9%	99.9%	100.0%	99.9%	99.9%

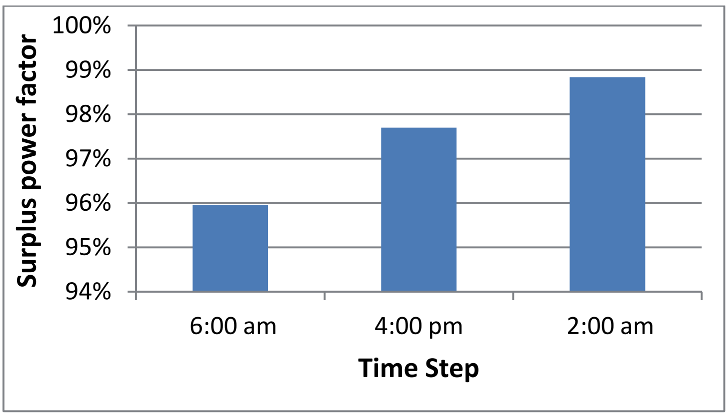

Figure 4.4 Average surplus power factor at a given time step

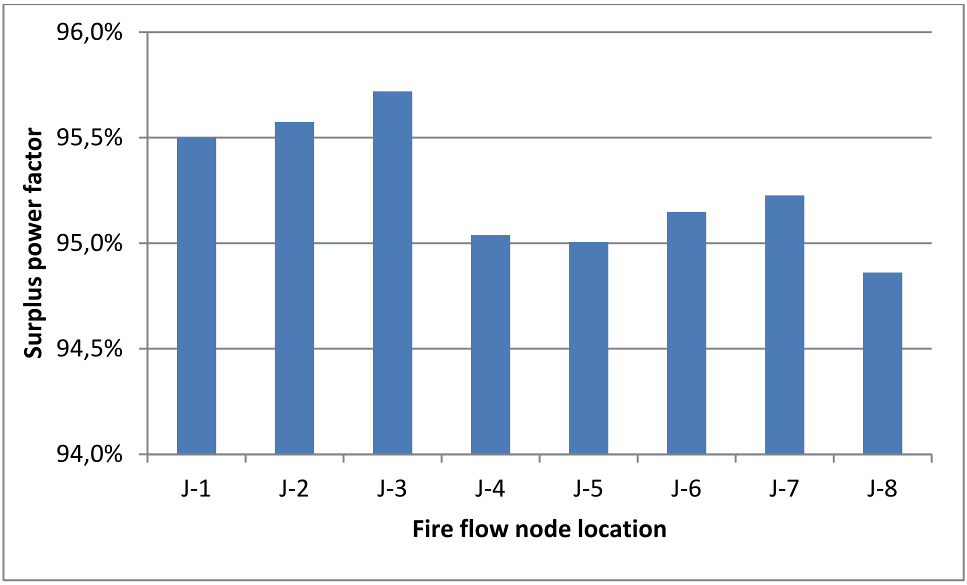

Figure 4.5 Average surplus power factor at 6:00 am + fire flow at a given node

As shown by the above results, s values are very high. Even in maximum demand conditions (6:00 am) and with fire flow applied, s does not fall below 94.9%. Most of the pipes have the s value well over 95%, while a smaller percentage of the pipes have the s value lower than 90% (data for each of the 2560 pipes are not presented here to save space).

The only explanation for such a result is that the pipe sections are rather short and the approach described in section 3.2 cannot be well applied to real world WDNs. The same hydraulic model could be skeletonised by reducing the number of nodes and increasing the length between the pipe sections. Thus, different results would be obtained with the same model. This approach could be applied to WTSs where long distances between the source and the target nodes are not interrupted by intermediate consumption nodes.

4.2.2. Calculation, version 2

In this case the network efficiency η_n and the surplus power factor s are calculated for the pipe system between the source and the target node.

The η_n and the s factor are calculated for some characteristic nodes around the WDN. Three nodes were selected for its maximum demand character; the other ones were selected for their position as the farthest from the source.

The simulation was conducted with the EPANET software. Input parameters to calculate η_n and s in the nodes were as follows:

- Total head at the input node (pumping station P1). Ground elevation in P1 is 22.0 meters and in P2 - 11 meters;
- Inflow into the system at the given time step at the pump house P1;
- Total head at the output node – the node where η_n and s were calculated.

The simulation was conducted for daily maximum, minimum and average demand conditions as well as for maximum demand condition in fire flow situation as in the version 1 analysis. It is clear that larger flows with fire scenarios increase the head loss in the WDS, therefore the s factor will be reduced.

The fire flow was analyzed with one fire event at a time at the same location where the surplus power factor s was calculated.

Firstly, the total heads at the pumping station P1 and the respective node as well as the inflow into the system were determined. Based on the head loss and inflow values, the network resistance coefficient C was determined with modified Eq. (2.32), see flowchart in Figure 3.3. In the case of ordinary WDSs the pressure loss (h) between the source and the target node is always lower than the input pressure (H). The network efficiency η_n was then calculated according to Eq. (2.40).

In order to calculate the coefficient of the critical output power k, firstly, $Q_{0\max}$ was determined. $Q_{0\max}$ was calculated based on Eq. (2.44) with the flow exponent a=2. After that k was determined according to Eq. (2.13. The surplus power factor was then calculated according to Eq. (2.14).

If η_n=2/3 or less, the calculated k reaches above its optimum point k=1 and starts to decrease again (i.e., s will increase), as seen in Figure 2.2. Therefore, if η_n=2/3 or less, the calculation results of k were ignored and s=0. Another parameter can be used to confirm the correct calculation of the s value. If Q_0>$Q_{0\max}$, then s has also reached its minimum value.

The abovementioned steps were performed with all the nodes in the system under maximum, average and minimum flow conditions. The fire flow was analyzed under a maximum demand condition (6:00 am).

The results for the network efficiency and the surplus power factor s calculations in the Õismäe-Mustamäe WDN are presented in tables (Table 4.3) to (Table 4.6). The results only in the respective nodes are presented in the tables. The distribution of the s value in all the nodes is presented in figures (Figure 4.8) to (Figure 4.11).

Denotations in the first column of the tables are referred as follows:

H_0, P1	Total head at pump station P1[m]
GL , J	Ground level at given node [m]
Q_0, P1	Inflow into pump station P1[L/s]
FF	Fire flow, L/s
Q_{0tot}	Total inflow into pump station P1 [L/s]
H_1, J	Total head at given node [m]
h	Head loss between source and target node [m]
C	Network resistance coefficient
η_n	Network efficiency
k	Coefficient of critical outlet power
s	Surplus power factor
Q_{0max}	Q_{0max}, L/s

Table 4.3 Network calculation results at 6:00 am

	J-1	J-2	J-3	J-4	J-5	J-6	J-7	J-8
H_0, P1	71.64	71.64	71.64	71.64	71.64	71.64	71.64	71.64
GL, J	21.27	23.90	25.96	13.89	35.57	7.06	8.61	11.90
Q_0, P1	229.3	229.3	229.3	229.3	229.3	229.3	229.3	229.3
H_1,J	63.05	66.23	69.79	61.07	62.86	61.31	61.30	60.84
h	8.59	5.41	1.85	10.57	8.78	10.33	10.34	10.80
C	0.00016	0.00010	0.00004	0.00020	0.00017	0.00020	0.00020	0.00021
$\boldsymbol{\eta_n}$	**88%**	**92%**	**97%**	**85%**	**88%**	**86%**	**86%**	**85%**
k	79%	66%	41%	85%	80%	84%	84%	86%
$\boldsymbol{s}$	**21%**	**34%**	**59%**	**15%**	**20%**	**16%**	**16%**	**14%**
Q_{0max}	382	482	824	345	378	349	348	341

Table 4.4 Network calculation results at 4:00 pm

	J-1	J-2	J-3	J-4	J-5	J-6	J-7	J-8
H_0, P1	71.50	71.50	71.50	71.50	71.50	71.50	71.50	71.50
Q_0, P1	140.2	140.2	140.2	140.2	140.2	140.2	140.2	140.2
H_1, J	68.99	69.81	70.85	68.47	68.87	68.43	68.43	68.42
h	2.51	1.69	0.65	3.03	2.63	3.07	3.07	3.08
C	0.00013	0.00009	0.00003	0.00015	0.00013	0.00016	0.00016	0.00016
$\boldsymbol{\eta_n}$	**96%**	**98%**	**99%**	**96%**	**96%**	**96%**	**96%**	**96%**
k	47%	39%	25%	51%	48%	52%	52%	52%
$\boldsymbol{s}$	**53%**	**61%**	**75%**	**49%**	**52%**	**48%**	**48%**	**48%**
Q_{0max}	432	526	849	393	422	391	391	390

Table 4.5 Network calculation results at 2:00 am

	J-1	J-2	J-3	J-4	J-5	J-6	J-7	J-8
H_0, P1	60.24	60.24	60.24	60.24	60.24	60.24	60.24	60.24
Q_0, P1	58.4	58.4	58.4	58.4	58.4	58.4	58.4	58.4
H_1, J	59.63	59.89	60.12	59.39	59.57	59.33	59.32	59.32
h	0.61	0.35	0.12	0.85	0.67	0.91	0.92	0.92
C	0.00018	0.00010	0.00004	0.00025	0.00020	0.00027	0.00027	0.00027
$\boldsymbol{\eta_n}$	**99%**	**99%**	**100%**	**99%**	**99%**	**98%**	**98%**	**98%**
k	26%	20%	12%	30%	27%	31%	32%	32%
$\boldsymbol{s}$	**74%**	**80%**	**88%**	**70%**	**73%**	**69%**	**68%**	**68%**
Q_{0max}	335	442	755	284	320	274	273	273

Table 4.6 Network calculation results at 6:00 am + fire flow

	J-1	J-2	J-3	J-4	J-5	J-6	J-7	J-8
H_0, P1	66.14	66.14	66.14	66.14	68.01	66.14	66.14	66.14
Q_0, P1	229.3	229.3	229.3	229.3	229.3	229.3	229.3	229.3
FF	**15.0**	**15.0**	**15.0**	**15.0**	**10.0**	**15.0**	**15.0**	**15.0**
Q_{0tot}	244.3	244.3	244.3	244.3	239.3	244.3	244.3	244.3
H_1, J	54.30	55.45	63.79	48.62	43.13	52.50	52.85	44.18
h	11.84	10.69	2.35	17.52	24.88	13.64	13.29	21.96
C	0.00020	0.00018	0.00004	0.00029	0.00043	0.00023	0.00022	0.00037
$\boldsymbol{\eta_n}$	**82%**	**84%**	**96%**	**74%**	**63%**	**79%**	**80%**	**67%**
k	90%	88%	47%	98%	100%	94%	93%	100%
$\boldsymbol{s}$	**10%**	**12%**	**53%**	**2%**	**0%**	**6%**	**7%**	**0%**
Q_{0max}	333	351	748	274	228	311	315	245

It can be seen that the η_n and s factor values vary from 85% to 100% and 14% to 88%, respectively, under normal diurnal demand conditions. In the fire flow condition, η_n and s values vary from 63% to 96% and from 0% to 53%, respectively. Unless the analyzed nodes are not too close to the source (i.e., J-3), the surplus power factor values under average demand conditions (4:00 pm) fluctuate around 40% to 50%. However, under fire flow conditions the same nodes show s values well below 10% and in some nodes the surplus factor approaches 0%. It could be seen that the farthest location has the smallest possibility to serve a large demand capability compared to the node that is close to the source. Under fire flow conditions, in particular, some nodes have surplus power at its minimum value (s=0).

The results for η_n and s values are plotted in figures (Figure 4.6) and (Figure 4.7). It is shown that under normal diurnal demand conditions the network is not as efficient as it could be, though Figure 4.6 shows that under fire flow conditions the η_n values are close to the minimum levels.

As Figure 4.6 shows, under fire flow conditions in some nodes η_n values tend to be smaller than 2/3. If η_n=2/3, the surplus power factor s reaches its minimum value (s=0) and has no capacity of further decrease. Therefore, it is required to calculate firstly η_n values and only then the coefficient of critical outlet power k and the s factor can be determined. If η_n value is skipped and s is calculated directly according to Eqs. (2.13) and (2.14), the results could be wrong since the minimum value of s could already be exceeded.

Thus, s depends significantly on local fire fighting regulations as well as city topology, consumption pattern and location of the pumping station. The case study demonstrates that under normal demand conditions s values show reasonable surplus capacity whereas under fire demand conditions it decreases practically to zero.

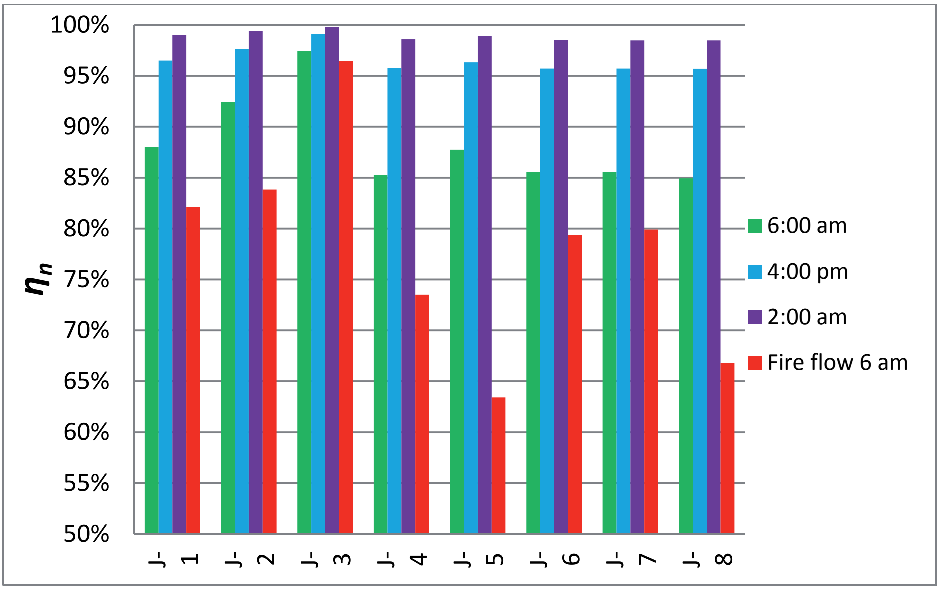

Figure 4.6 Coefficient of network efficiency at a given hour

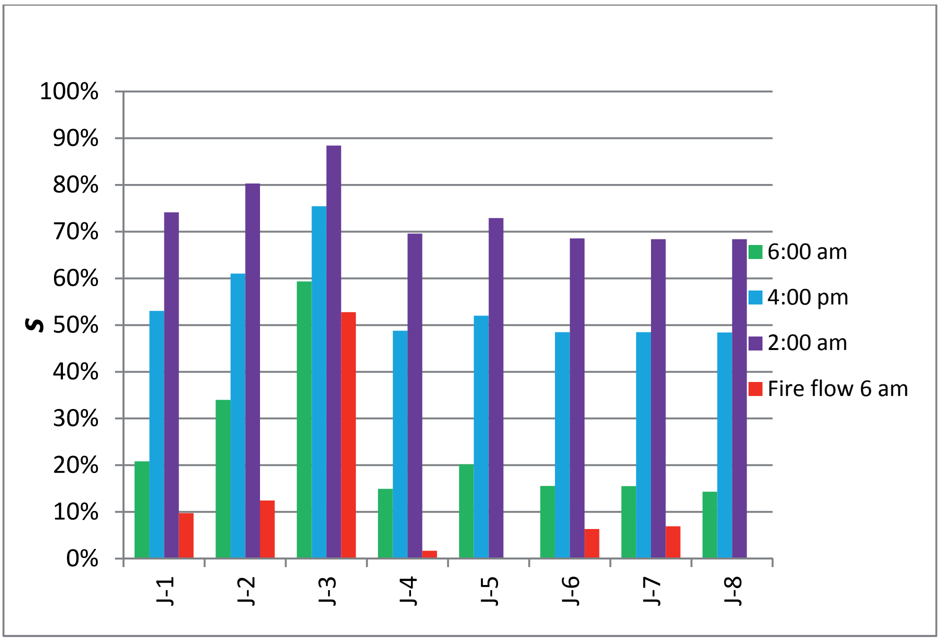

Figure 4.7 Surplus power factor at network nodes at a given hour

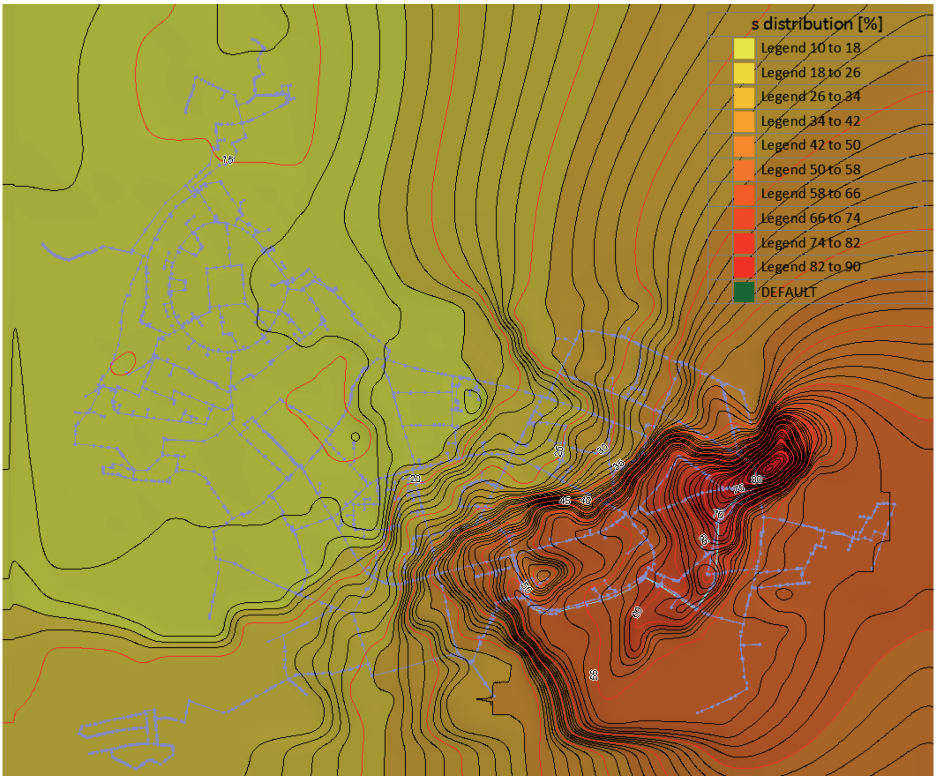

Figure 4.8 Distribution of *s* values around the Õismäe-Mustamäe WDN at the maximum demand condition (6:00 am)

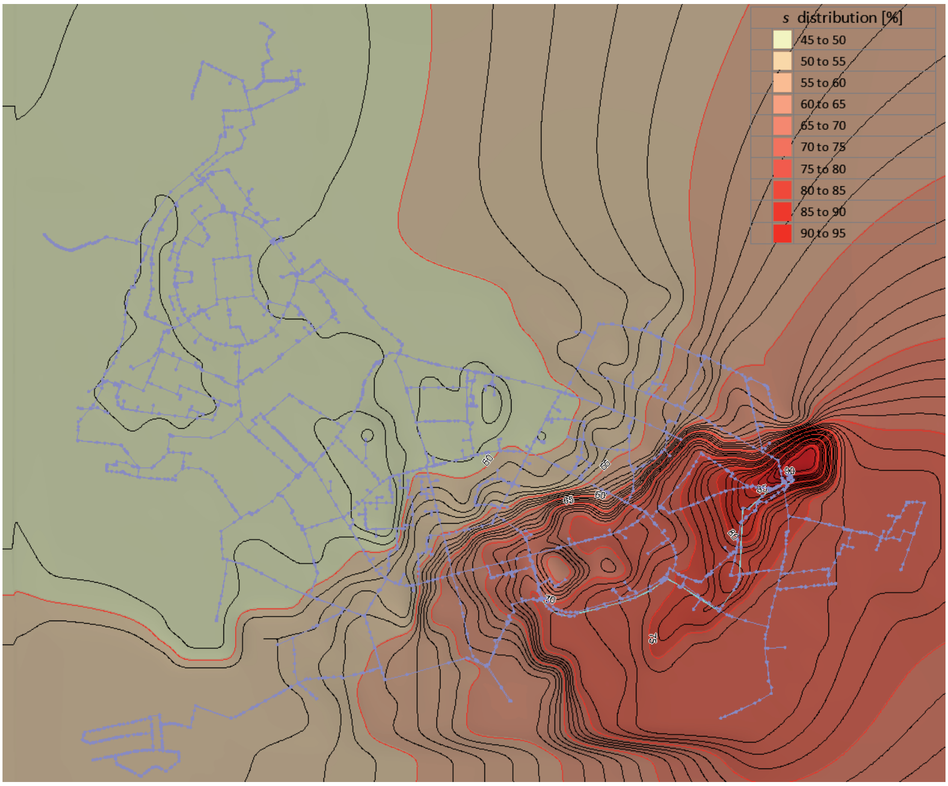

Figure 4.9 Distribution of *s* values around the Õismäe-Mustamäe WDN at an average demand condition (4:00 pm)

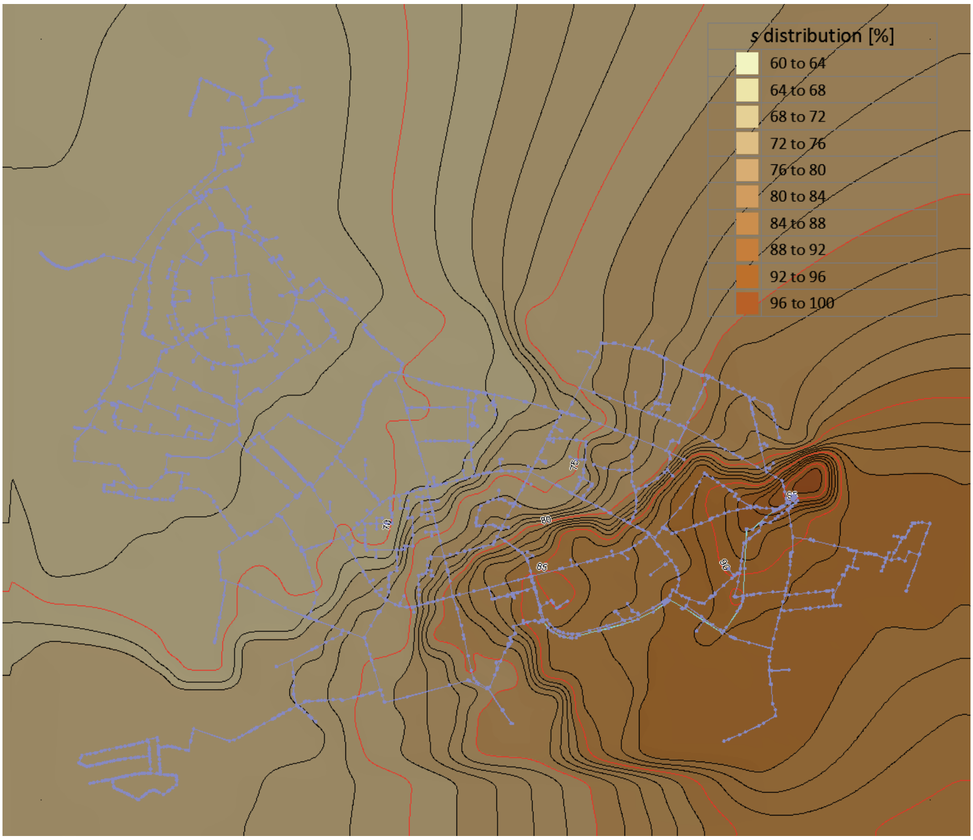

Figure 4.10 Distribution of *s* values around the Õismäe-Mustamäe WDN at the minimum demand condition (2:00 am)

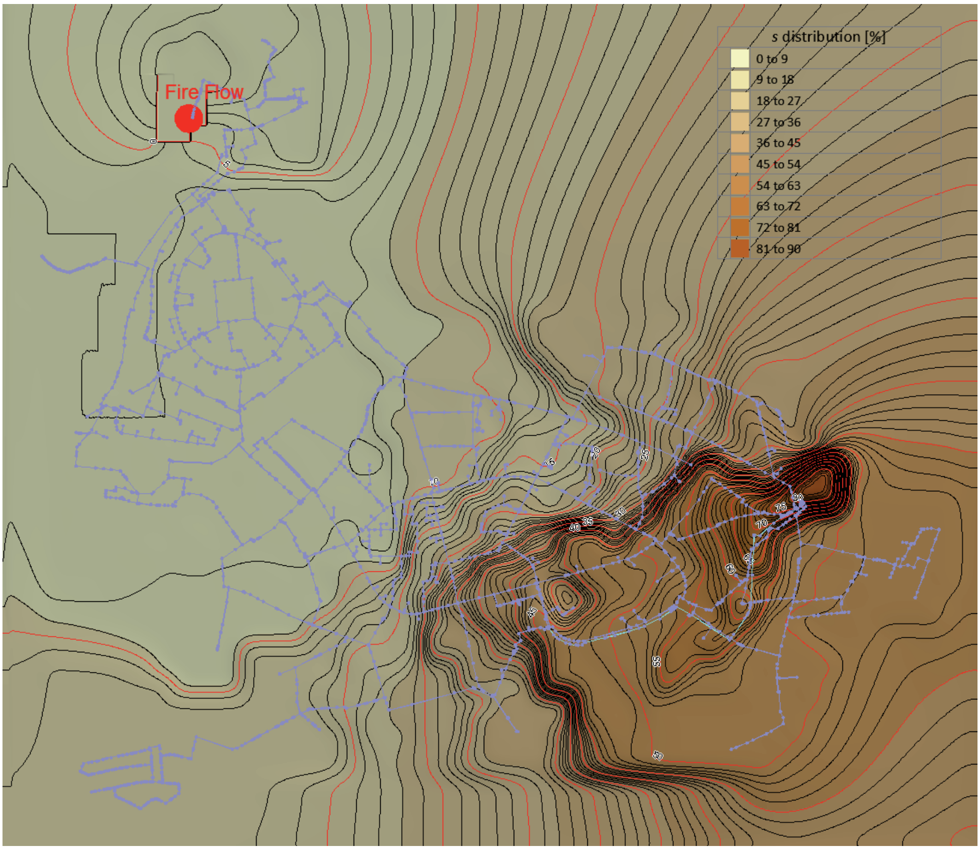

Figure 4.11 Distribution of *s* values around the Õismäe-Mustamäe WDN at the maximum demand condition with fire flow 15 L/s

4.2.3. Calculation version 3

According to version 3, the initial data for calculations are the same as for previous versions. The hydraulic model was solved with EPANET.

Firstly, the analysis results for all pipes (flow, length, diameter and friction factor) were exported and head loss in each pipe was calculated. Based on Eq. (2.37), the resistance coefficient of the water network was calculated. In order to calculate the coefficient of the critical output power *k*, $Q_{0\max}$ was determined by accounting the total head at the pumping station (Eq. 2.44). After that *k* was determined according to Eq. (2.13) with the flow exponent a=2. The surplus power factor was then calculated according to Eq. (2.14). The network efficiency η_n was calculated according to Eq. (2.40).

The above steps were performed for different demand conditions and with all the nodes J-1 to J-8 under fire flow conditions. The results are presented in Table 4.7 and Table 4.8.

Table 4.7 Calculation results of the surplus power factor at different time steps

	6:00am	4:00pm	2:00am
C	**68.7**	**40.9**	**174.2**
Q_0, L/s	285.38	198.37	58.51
H_0	71.64	71.53	60.24
P_0, kW	200.36	139.06	34.54
P_d, kW	15.65	3.13	0.34
P_u, kW	184.71	135.93	34.20
Q_{0max}	589.6	763.5	339.5
s	33%	62%	74%
η_n	92.2%	97.7%	99.0%

Table 4.8 Calculation results of the surplus power factor at 6:00 am + fire flow at a given node

	J-1	**J-2**	**J-3**	**J-4**	**J-5**	**J-6**	**J-7**	**J-8**
C	75.7	71.8	62.4	81.4	79.0	78.6	78.4	123.0
Q_0, L/s	285,38	285,38	285,38	285,38	285,38	285,38	285,38	285,38
FF, L/s	15,0	15,0	15,0	15,0	10,0	15,0	15,0	15,0
Q_{0tot}, L/s	300.38	300.38	300.38	300.38	295.38	300.38	300.38	300.38
H_0	66.14	66.14	66.14	66.14	70.02	66.14	66.14	66.14
P_0, kW	194.70	194.70	194.70	194.70	202.69	194.70	194.70	194.70
P_d, kW	20.11	19.07	16.57	21.61	19.94	20.88	20.88	32.67
P_u, kW	174.59	175.63	178.13	173.09	182.74	173.82	173.82	162.02
Q_{0max}	539.7	554.2	594.5	520.6	543.7	529.6	529.6	423.3
s	25%	27%	31%	23%	27%	24%	24%	11%
η_n	89.7%	90.2%	91.5%	88.9%	90.2%	89.3%	89.3%	83.2%

It can be seen that the surplus power factor s values vary from 33% to 74% and the network efficiency η_n from 92% to 99% under normal diurnal demand conditions. In the fire flow condition s values vary from 11% to 31% and η_n from 83% to 92%, respectively.

As shown, the farthest location has the lowest possibility to serve a large demand capability compared to the node that is close to the source. Under fire flow conditions, in particular, η_n and s values tend to be smaller than those with the fire flow nearer to the source node, for example, nodes J-3 and J-8, since the head loss through the pipes is increased significantly between the source and the target nodes.

The results for η_n and s values are plotted in Figure 4.12 and Figure 4.13. At all conditions, η_n values are higher than 2/3 (Figure 4.12). If η_n=2/3, the surplus power factor s reaches its minimum value (s=0) and has no capacity to decrease further.

The case study demonstrates that under normal demand conditions s values show reasonable surplus capacity whereas under fire demand conditions it decrease significantly depending on fire flow demand and location in the WDN. To propose certain s values for new design or rehabilitation of existing WDSs, different types of WDS should be studied. At maximum demand conditions, the recommended s value for a WDN should have a minimum value of at least 20%. This will allow the WDN to operate properly in fire flow conditions. This recommendation is based on Estonian fire regulations that determine the minimum pressure of 100 kPa in a WDN. However, if larger fire flows are required, the minimum recommended surplus power factor value should also be reconsidered.

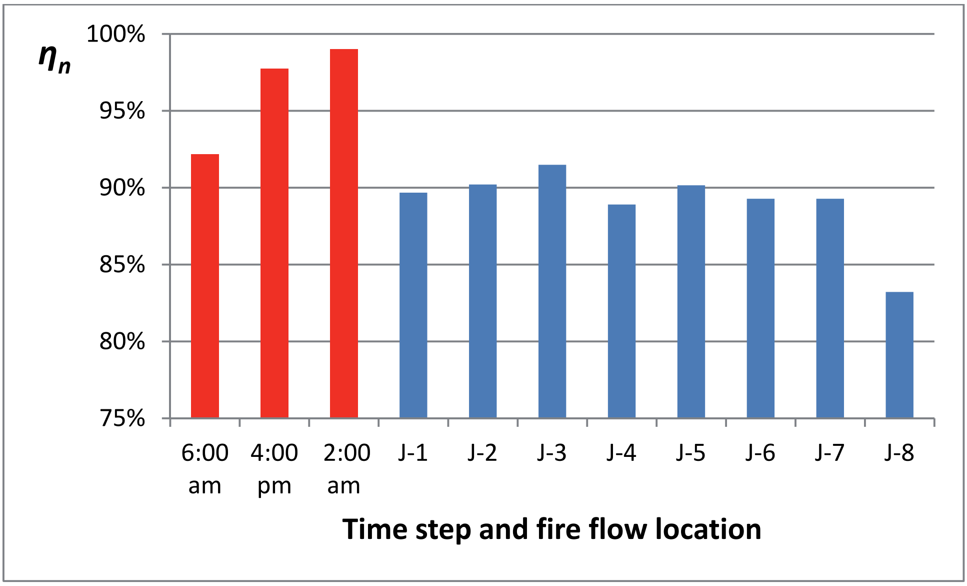

Figure 4.12 Hydraulic efficiency of the network at a given hour and given fire flow location (6:00 am)

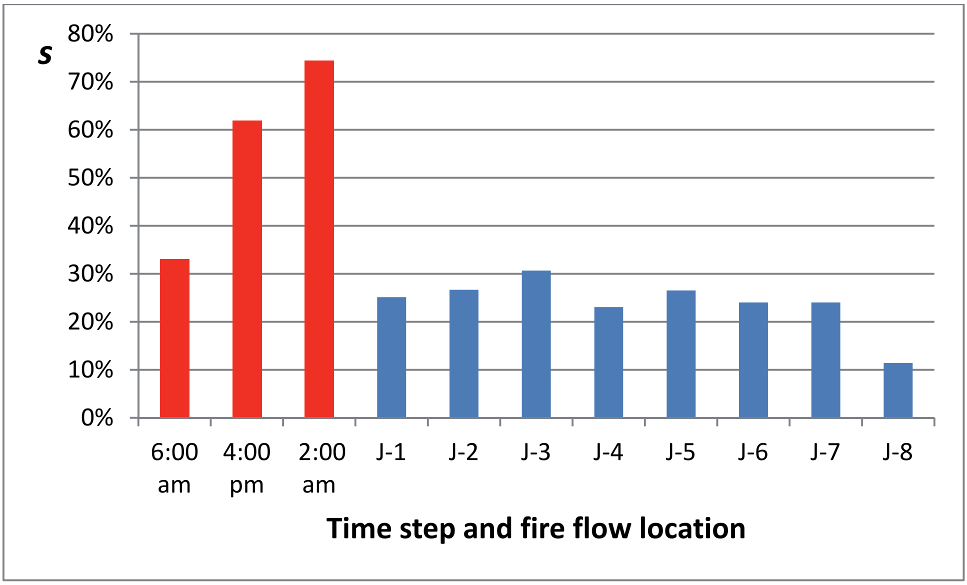

Figure 4.13 Surplus power factor at network nodes at a given hour and given fire flow location (6:00 am)

4.2.4. Comparison of calculation results for versions 1, 2 and 3

The results of the three different calculation approaches vary considerably.

The main difference lies in the method of determination of the network resistance coefficient *C*. The deficiencies of version 1 revealed that the short pipe sections that are independently contributing to the average surplus power factor of the system do not contribute to the head loss in a manner that could be used in real world WDNs. On the contrary, the calculation according to version 2 revealed that the path between the source and the target node could be too long so that even a small change in demands (i.e. fire flow) could contribute to pressure loss that is fair enough for a large *s* value drop.

As a result, version 3 as the correct method was further developed to determine the network resistance coefficient that could be well applied to surplus power factor analysis.

The results calculated using one of the three approaches are compared in Figure 4.14 and Figure 4.15. Version 1 and 3 represent the average *s* value for WDN while for version 2 the node J-1 was selected (Figure 4.14). To compare fire flow results, node J-1 was selected for all versions (Figure 4.3).

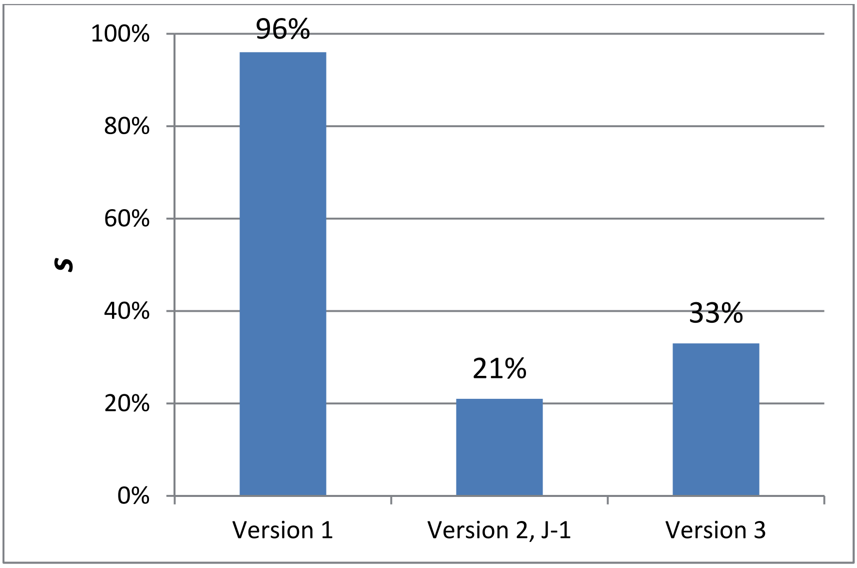

Figure 4.14 Comparison of *s* values of three different calculation methods (6:00 am).

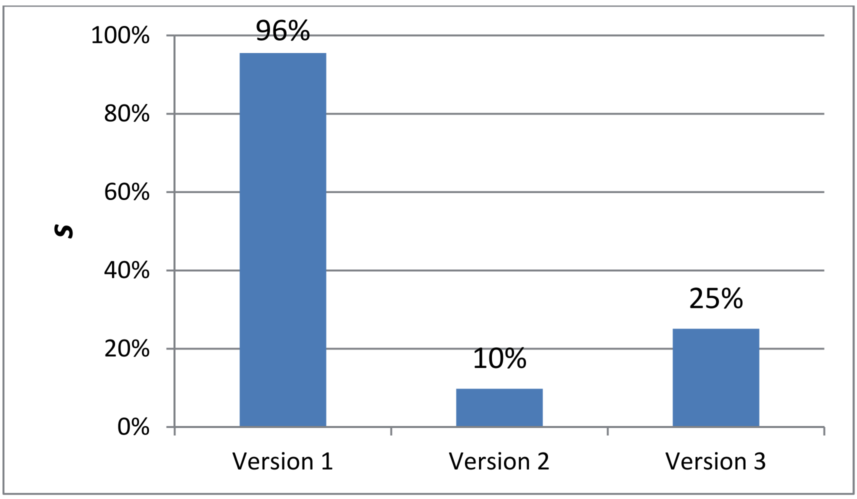

Figure 4.15 Comparison of *s* values of three different calculation methods (node J-1, fire flow 6:00 am).

The other network locations and time steps would give different results, but the overall idea will be the same - version 1 approach will give too high surplus values, while version 2 results tend to give rather low values, especially in fire flow conditions. The results for version 3 tend to find the balance between previous two since it accounts for overall network behaviour during the change of demand conditions in some node of the system.

4.2.5. Õismäe-Mustamäe WDN reconfigured

An application for the rehabilitation program of existing WDSs is to use an optimisation strategy of pipe diameters. Since the majority of the pipelines in Õismäe-Mustamäe WDN are older than 30 years, one approach could be changing pipe diameters during the rehabilitation process. There is a myriad of different optimisation algorithms for WDNs, but the optimisation topic is out of scope of this thesis and instead, the following three simple scenarios are considered:

- **Case 1**. Pipes with diameters 100 to 250 mm have been increased by one nominal diameter. This group forms the majority of pipes in Õismäe-Mustamäe WDN (see Figure 4.1)
- **Case 2.** Pipes with diameters 400 mm and larger have been reduced by one nominal diameter
- **Case 3.** Pressure in pump house P1 has been reduced by approximately 1 bar

A question may arise about the reason to apply the surplus power factor analysis. Therefore, the approach using calculation version 3 was tested on simple optimisation cases as described above.

Firstly, in Case 1 the pipe diameters in the Õismäe-Mustamäe WDN model were replaced according to Table 4.9. Then, the *s* analysis was conducted according to section (4.2.3).

Table 4.9 Pipe diameter change in the hydraulic model in Case 1.

Initial diameter, mm	New diameter, mm
90	150
94	150
100	150
110	150
136	175
150	200
160	200
200	250

Secondly, in Case 2 the pipe diameters in the Õismäe-Mustamäe WDN model were replaced according to Table 4.10. Diameters for pipes 100 to 250mm were left unchanged. Again, the *s* analysis was performed according to section (4.2.3).

Table 4.10 Pipe diameter change in the hydraulic model in Case 2

Initial diameter, mm	**New diameter, mm**
400	350
500	450
530	450

Finally, the pump curve for P1 was modified so that the pressure in the pump house P1 was lowered approximately by 1 bar. The pipe diameters were left unchanged.

The results for the three cases are presented in tables (Table 4.11) to (Table 4.13)

Table 4.11 Pipe diameters 100÷250mm increased by one nominal diameter

	6:00	16:00	2:00
C	**47.3**	**29.0**	**115.1**
Q_0, **L/s**	285.38	198.37	58.52
H_0	71.64	71.53	60.24
P_0, **kW**	200.36	139.06	34.55
P_d, **kW**	10.76	2.22	0.23
P_u, **kW**	189.59	136.84	34.32
Q_{0max}	710.9	906.3	417.6
s	43%	68%	79%
η_n	94.6%	98.4%	99.3%

Table 4.12 Pipe diameters >400mm decreased by one nominal diameter

	6:00	16:00	2:00
C	**82.6**	**50.1**	**201.9**
Q_0, **L/s**	285.38	198.37	58.52
H_0	71.64	71.53	60.24
P_0, **kW**	200.36	139.06	34.55
P_d, **kW**	18.81	3.83	0.40
P_u, **kW**	181.55	135.22	34.15
Q_{0max}	537.8	689.9	315.4
s	28%	58%	72%
η_n	90.6%	97.2%	98.9%

Table 4.13 Pressure in the pumping station P1 reduced by 1 bar

	6:00	16:00	2:00
C	68.7	40.9	174.2
Q_0, **L/s**	285.38	198.37	58.52
H_0	60.10	60.01	51.35
P_0, **kW**	168.08	116.66	29.45
P_d, **kW**	15.65	3.13	0.34
P_u, **kW**	152.44	113.53	29.11
Q_{0max}	540.0	699.3	313.5
s	28%	59%	72%
η_n	90.7%	97.3%	98.8%

The comparison of the initial model versus those "optimized" is presented in Figure 4.16

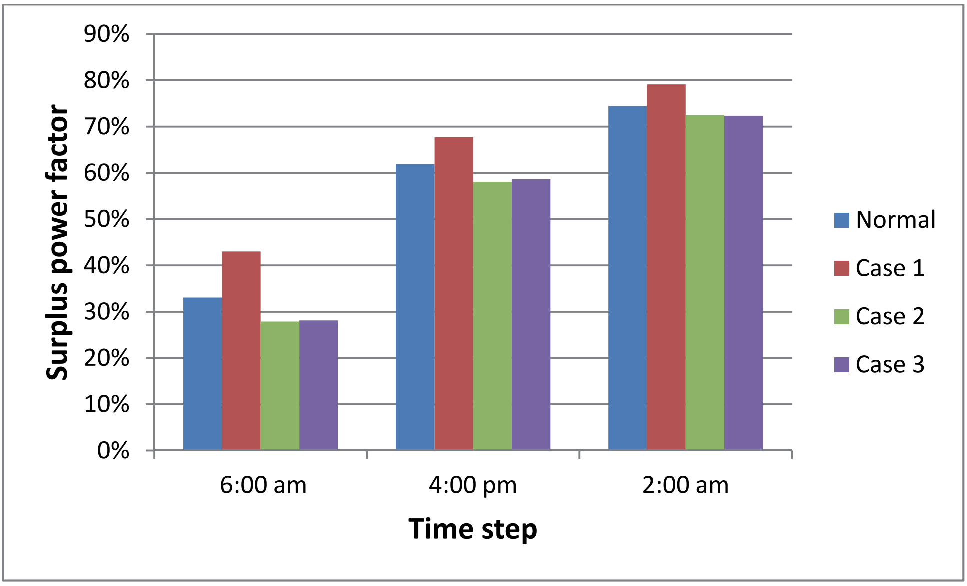

Figure 4.16 Surplus power factor values at different WDN configuration scenarios

In a maximum demand condition, the change of *s* value is notably higher if the majority of the pipe diameters are increased (Case 1). However, for average or minimum demand conditions the *s* value does not increase on a large scale. If the capacity of the WDN is reduced (Cases 2 and 3), *s* will decrease compared to initial network characteristics. Still, the reduction is rather small and *s* value is in reasonable limits even in the maximum demand condition.

In conclusion, the overall configuration of the Õismäe-Mustamäe WDN could cope with the diameter changes or variations in the pressure without losing its resilience.

4.3. Surplus Power Factor Analysis of the Viimsi Water Transmission System

Case study 2 was performed on a WTS that delivers groundwater from bore wells to a water treatment plant reservoir. The difference from a WDN is that no consumption nodes exist between the source and target nodes and water is delivered with a constant demand pattern.

Viimsi WTS is located near Tallinn and delivers water for more than 20000 customers. The layout of the WTS is presented in Figure 4.17.The WDN that delivers water from the treatment plant to consumers is not under the scope in this thesis.

The bore well group consists of 5 bore wells. At each bore well location groundwater is pumped from two separate water aquifers, one of which has better quality (Voronka layer) while the other one (Gdov layer) has better discharge rate. The piping system from the bore wells is parallel up to the water treatment plant where different quality waters are mixed. The water treatment consists of removal iron and radioactive compounds. The water treatment plant is located on the highest level compared to the WDN. Therefore, approximately 2/3 of the consumers could be supplied with treated water without the need for pumping. That guarantees the delivery of water in case of emergencies (i.e. fire fighting purposes) even in total blackout conditions.

The distance from the bore well group to the water treatment plant is approximately 2.5 km. The total capacity of the water treatment plant is 6000 m^3/d. Figure 4.18 presents the layout of the Viimsi WTS hydraulic (EPANET) model.

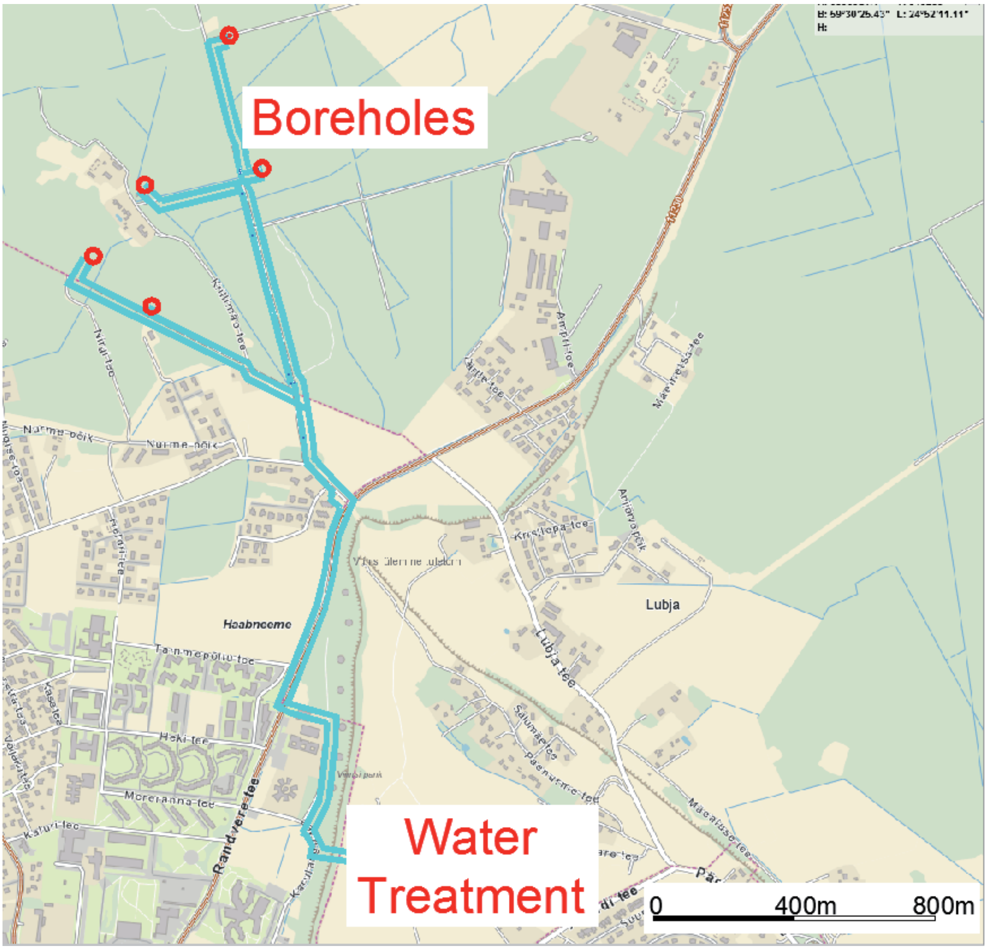

Figure 4.17 Viimsi WTS layout

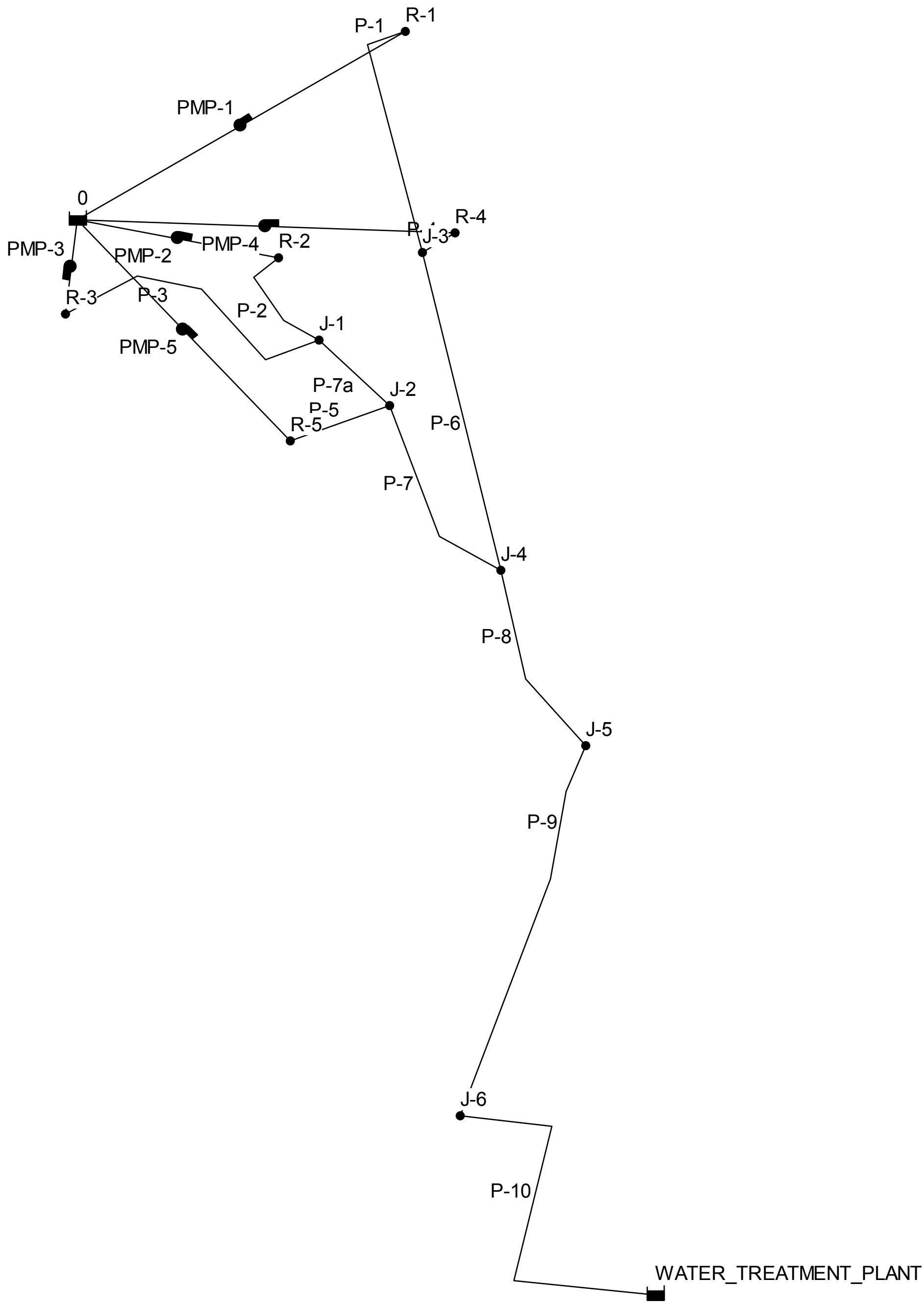

Figure 4.18 Hydraulic model of the Viimsi WTS (nodes and pipes)

The network pipe data are presented in Table 4.15. For Gdov and Voronka groundwater layers, two separate hydraulic models were analyzed. Pipe lengths and pumps locations were the same, only pipe diameters and pump characteristics were different.

Pipe selection in the model is based on the manufactured PE pipes according to Table 4.14

Table 4.14 Manufactured pipe diameters

Pipe outside diameter, mm	Pipe inside diameter, mm
110	97
160	141
200	178
225	198
250	213
280	240
315	278

Table 4.15 Pipe characteristics of Viimsi WTS

Pipe ID	Length, m	Pipe inside diameter, mm	
		Gdov layer	Voronka layer
P-1	474.9	141	97
P-2	229.2	141	97
P-3	553.8	141	97
P-4	70.7	141	97
P-5	195.7	141	97
P-6	611.1	198	141
P-7	392.6	213	141
P-7a	178.6	198	141
P-8	375.5	240	178
P-9	729.7	240	178
P-10	731.8	240	178

Pump characteristics and efficiency curves are presented in Figure 4.19 and Figure 4.20. In each pump station location there are two bore wells with one pump.

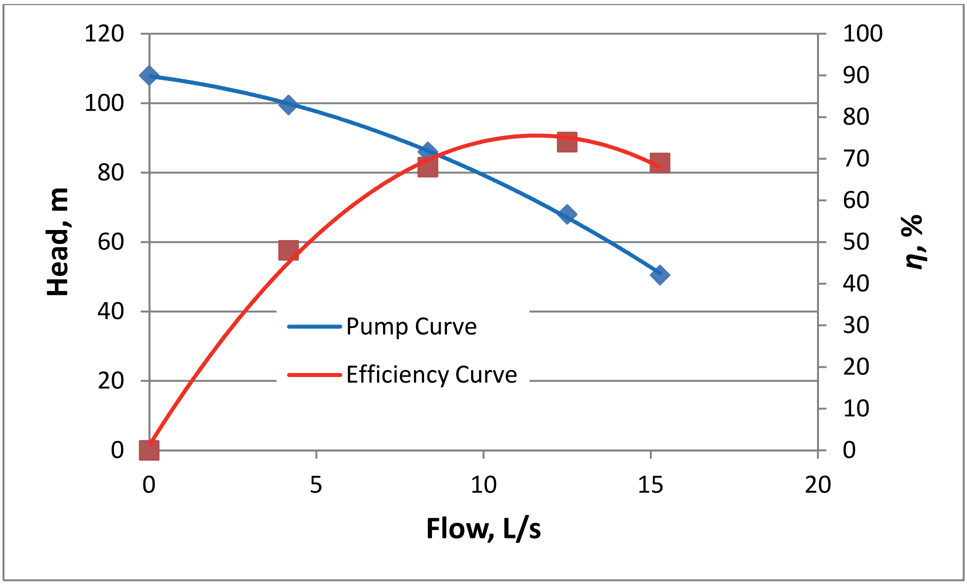

Figure 4.19 Pump characteristics and efficiency curves for the Gdov layer

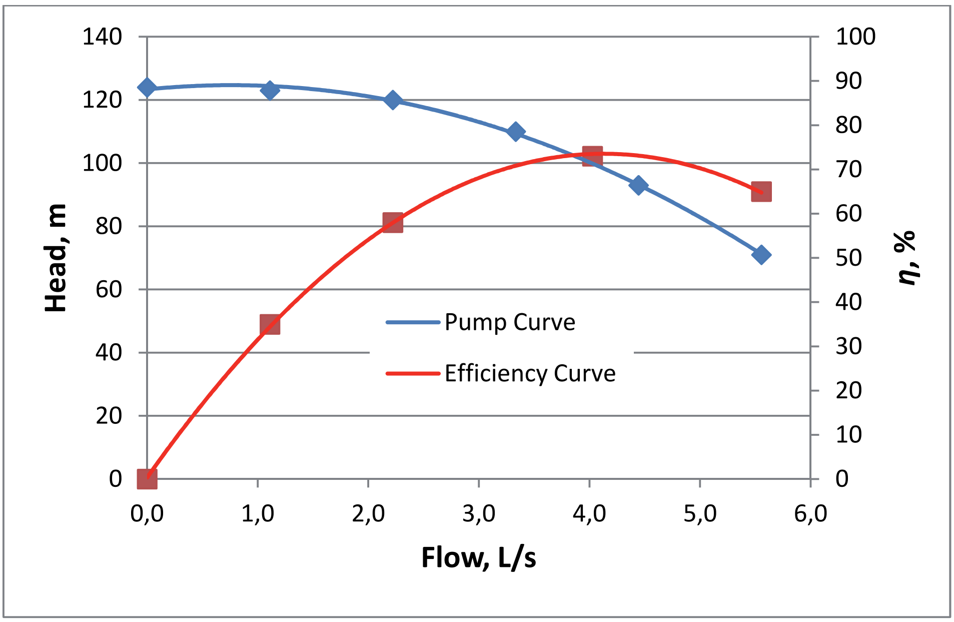

Figure 4.20 Pump characteristics and efficiency curves for the Voronka layer

The analysis was conducted considering two scenarios for the Gdov and Voronka models:

- All five pumps running
- Three pumps running (PMP-1, PMP-2 and PMP-4)

4.3.1. Hydraulic simulation results

The hydraulic steady state simulation results are presented in Table 4.16 and Table 4.17. For each pump the efficiency was determined according to the characteristic curves of the pump. In order to determine the overall efficiency η of pumping as presented in Eq. (2.48), the efficiencies of all working pumps were averaged.

Table 4.16 Hydraulic simulation results for the Voronka layer

Link ID	5 pumps running			3 pumps running		
	Flow LPS	Velocity m/s	Pump efficiency	Flow LPS	Velocity m/s	Pump efficiency
Pipe P-1	5.25	0.71		5.51	0.75	
Pipe P-2	5.26	0.71		5.63	0.76	
Pipe P-3	5.18	0.7		0.00	0.00	
Pipe P-4	5.36	0.72		5.61	0.76	
Pipe P-5	5.29	0.72		0.00	0.00	
Pipe P-6	10.61	0.68		11.13	0.71	
Pipe P-7	15.73	1.01		5.63	0.36	
Pipe P-7a	10.43	0.67		5.63	0.36	
Pipe P-8	26.34	1.06		16.76	0.67	
Pipe P-9	26.34	1.06		16.76	0.67	
Pipe P-10	26.34	1.06		16.76	0.67	
Pump PMP-3	5.18		69	0.00		
Pump PMP-2	5.26		68	5.63		64
Pump PMP-5	5.29		68	0.00		
Pump PMP-4	5.36		67	5.61		64
Pump PMP-1	5.25		68	5.51		65

Table 4.17 Hydraulic simulation results for the Gdov layer

Link ID	5 pumpsrunning			3 pumpsrunning		
	Flow LPS	Velocity m/s	Pump efficiency	Flow LPS	Velocity m/s	Pump efficiency
Pipe P-1	11.07	0.71		12.0	0.77	
Pipe P-2	11.17	0.72		12.4	0.79	
Pipe P-3	10.93	0.7		0.0	0.00	
Pipe P-4	11.34	0.73		12.3	0.79	
Pipe P-5	11.27	0.72		0.0	0.00	
Pipe P-6	22.41	0.73		24.2	0.79	
Pipe P-7	33.38	0.94		12.4	0.35	
Pipe P-7a	22.11	0.72		12.4	0.40	
Pipe P-8	55.79	1.23		36.7	0.81	
Pipe P-9	55.79	1.23		36.7	0.81	
Pipe P-10	55.79	1.23		36.7	0.81	
Pump PMP-3	10.93		75	0.0		
Pump PMP-2	11.17		75	12.4		75
Pump PMP-5	11.27		75	0.0		
Pump PMP-4	11.34		76	12.3		75
Pump PMP-1	11.07		75	12.0		75

4.3.2. Results of surplus power factor analysis

In addition to surplus power factor s and network efficiency coefficient η_n, two additional parameters were determined in the Viimsi WTS analysis – pumps (η) and system efficiencies (η_s). This allowed an analysis of the pump and network in joint operation.

The network resistance coefficient C, surplus power factor s and network efficiency coefficient η_n were determined as described in section (4.2.3). The hydraulic power (P_0, P_d and P_u) for the network was determined according to Eq. (2.33). The hydraulic power at the inlet of the system, P_0, was determined according to the inflow and total head into the system. In this case Q_0 is the sum of the discharge rate of all the pumps. For H_0 the total head for either 3 or 5 pumps was averaged. Pump station efficiencies were averaged for five or three pumps, respectively. The system efficiency was calculated according to Eq. (2.48)

The results for the Viimsi WTS system analysis are presented in Table 4.18.

Table 4.18 Results of surplus power factor and system efficiency analysis for the Viimsi WTS

	Voronka layer		Gdov layer	
	5 pumps	3 pumps	5 pumps	3 pumps
C	20100	25866	3665	4706
Q_0, L/s	26.34	16.76	55.79	36.65
H_0	67.74	60.21	65.67	59.64
P_0, kW	17.49	9.89	35.90	21.42
P_d, kW	3.60	1.19	6.24	2.27
P_u, kW	13.89	8.70	19.15	19.15
Q_{0max}	33.5	27.9	77.3	65.0
s	6.4%	20.6%	10.5%	24.4%
η_n	79.4%	87.9%	82.6%	89.4%
Pump stations				
η	68%	64%	75%	75%
P	25.74	15.35	47.59	28.44
η_s	54%	57%	62%	67%

As can be seen from Table 4.18, the overall system efficiency η_s for the Voronka model is higher, in case 3 pumps are running although in this case the efficiency of pumps is lower than 5 pumps running. This can be explained by the fact that in case 3 the network efficiency η_n of the pumps is much higher than that of 5 pumps running, which contributes to higher system efficiency value. Therefore, it is always worth analyzing pumps and network in joint operation.

4.4. Multi-objective Trade-off

The optimization of WDS is often regarded as solving of a multi-objective (MO) task. Highly optimal solutions can be found to transport water from point A to B, but this would definitely not fulfil the customers' needs who get the water from tap even in case of unexpected failures in the system. Therefore, designers often add some safety factor in order to cope with unexpected system behaviour, which makes the system more expensive - the more you pay the more you get. It is just the balance of cost versus optimal solution that needs to be found.

The optimization of WDSs is not under the scope in this thesis. The idea here is to compare some of the results that could be obtained from a multi-objective trade-off analysis and surplus power factor analysis.

The following section (4.4.1) is direct quotation from WaterGEMS (Bentley 2010) user manual and describes the procedure of the multi-objective trade-off analysis that can be performed with WaterGEMS software (Darwin Designer module).

4.4.1. Description of WaterGEMS software trade-off analysis

The benefit of the hydraulic performance is measured by using junction pressure (p) improvements. Two types of pressure benefit are provided in Darwin Designer, namely dimensionless benefit and unitized benefit.

Dimensionless Pressure Benefit

The pressure improvement for a dimensionless benefit is proposed as a ratio of pressure difference between the actual pressure and a user-defined reference pressure. The benefit is normalized by the junction demand (*JQ*). The factors are also introduced to enable a modeller to convert and customize the hydraulic benefit function.

$$HYBenefit = \sum_{k=1}^{ND} \left\{ a \sum_{i=1}^{NJ} \left(\frac{JQ_{i,k}}{JQtotal_k} \right) \left[\frac{\left(p_{i,k} - p_{i,k}^{ref} \right)}{p_{i,k}^{ref}} \right]^b \right\} \tag{4.1}$$

where a and b are factors that allow an optimization modeller to weigh, convert, and customize pressure improvement to hydraulic benefit; NJ is the number of pressure benefit junctions; ND is the number of design events for which the pressure benefit is considered; $JQ_{i,k}$ is a demand at junction i for demand alternative k; $JQtotal_k$ is a total junction demand for alternative k; $p_{i,k}$ is a post-rehabilitation pressure at junction i for demand alternative k; p^{ref} is a reference junction pressure defined by a user to evaluate the pressure improvement. The reference pressure is taken as the minimum required junction pressures.

Unified Pressure Benefit

Pressure benefit resulting from a design and rehabilitation can also be quantified by using the unitized average pressure improvement across the entire system. The unified pressure benefit functions can be given as follows:

$$p_{avg} = \sum_{k=1}^{ND} \frac{\sum_{i=1}^{NJ} \left| p_{i,k} \right| - \left| p_{i,k}^{ref} \right|}{NJ} . \tag{4.2}$$

The advantage of using the unitized pressure benefit function is that a modeller is able to evaluate the average pressure enhancement for the investment. It is worth being aware of the value of the Euros spent.

4.4.2. Trade-off analysis of the Õismäe-Mustamäe WDN

This section compares the trade-off analysis of commercial software and the surplus power factor analysis. WaterGEMS software was selected as it allows one to create multiple WDN configuration scenarios based on user specified maximum cost and pressure benefit. The result is that the software generates for each scenario its own WDN model that can be exported to user selectable file formats. In this case the scenarios generated by WaterGEMS were exported to EPANET format and it allowed calculations necessary for the surplus power factor analysis. MO trade-off analysis was carried out for the Õismäe-Mustamäe WDN described in section (4.2.3).

In order to perform MO optimization task with WaterGEMS Darwin Designer module initial parameters have to be set as follows:

- The selection of the pipes changed during the optimization process. In order to perform the optimization in a reasonable amount of time and avoid the optimization of non-recommended pipe sections (i.e., dead ends, house connections etc), certain WDN sections are selected. In this case mains between pump house and farthest node were selected as presented in Figure 3.2.
- The cost of pipe for different diameters must be defined, as in Table 4.19
- The maximum cost of optimized pipes. This will limit the possible scenarios that software will generate. In this case 4 million Euros were set as a maximum "budget". This is the setting for the pipe sections that are optimized (Figure 3.2). The analysis considers the cost of all pipes in the Õismäe-Mustamäe WDN.

Table 4.19 Pipe diameters and unit costs for the Õismäe-Mustamäe WDN

Diameter	€/m
95	50
100	55
110	60
135	70
150	84
160	95
200	128
250	173
255	178
270	186
300	231
315	238
325	245
350	269
400	358
450	428
500	499
530	512

Based on the parameters previously described, WaterGEMS generated 18 different scenarios. The results of the analysis are presented in Figure 4.21 and Figure 4.22.

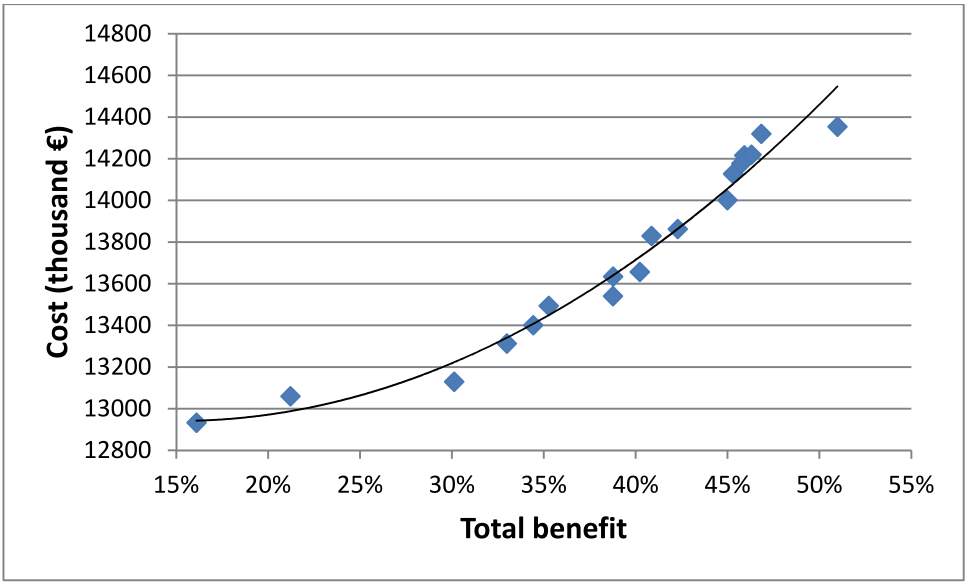

Figure 4.21 The cost of WDN versus WaterGEMS' Total benefit

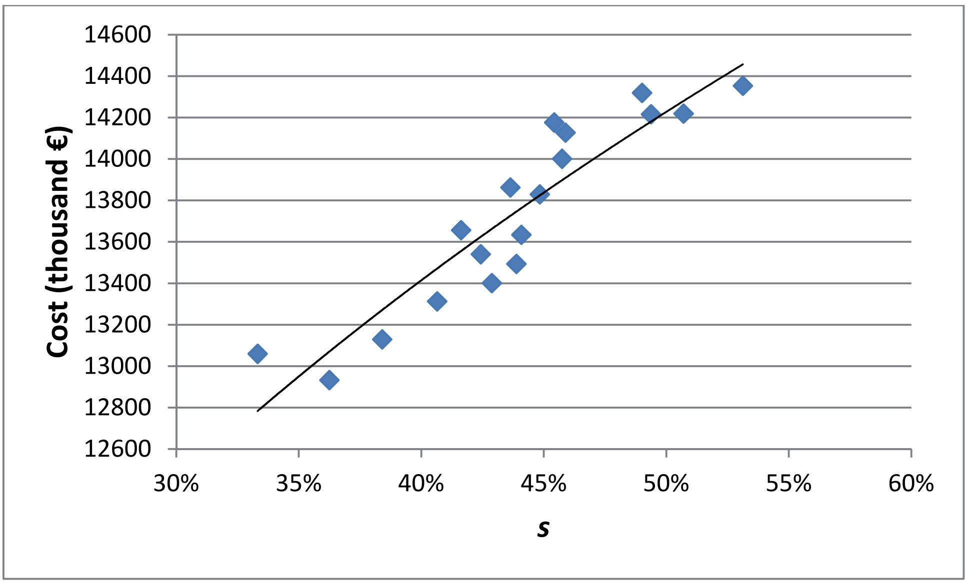

Figure 4.22 The cost of the Õismäe-Mustamäe WDN versus the surplus power factor

Each dot represents the cost versus the total benefit or the surplus power factor value of each scenario. The increase of total benefit values follows the cost trend line. Although the WaterGEMS software is not intended for analyzing the *s* factor versus the cost, it can be used to generate different cost alternatives and hydraulic model scenarios and these can be used in the *s* factor analysis with EPANET software. Figure 4.22 shows the cost versus the *s* factor results.

5. CONCLUSIONS AND FURTHER RESEARCH

Focus in this thesis is on the hydraulic power capacity and energy transmission in WDSs. As a result, the surplus power factor s and the coefficient of the network efficiency η_n, were developed that are new integral parameters in the reliability analysis of the WDNs.

Chapter 1 covers the historical overview of the development of reliability in WDSs. Research in the 1990s was especially fruitful. After the energy prices rose sky-high just before the economic recession during the first decade of the 21st century, several articles were published.

Chapter 2 describes in detail the development of the reliability index of the surplus power factor, followed by the chapters were case studies for an in-service WDS are presented. Few studies have analyzed the hydraulic power capacity of real water networks. Different theoretical approaches show that this method can be used to characterize the reliability of a WDS (Wu et al. 2011). Although the results from different authors explain it as one method to characterize the reliability of the systems, it is sometimes complicated to apply their findings into everyday practice.

5.1. Summary of findings

Two case studies, one for a WDS, and the other for a WTS were conducted. The first case study for the Õismäe-Mustamäe WDN (2480 nodes and 2560 pipes) reveals how different selections of the calculation algorithms affect s values. The development of the network matrix equations revealed that the approach that uses one overall network resistance coefficient C for the whole WDN gives most applicable results for s and η_n analysis (version 3).

In section (4.2.3) the results for the s analysis are presented. Factor s value for the Õismäe-Mustamäe WDN network is 33%, 62%, and 74% for maximum, average, and minimum daily demand condition. In case of fire flow the s value varies from 11% to 31%, depending on the location of the fire flow. Based on the results, the minimum recommended s value should be above 20% limit. This will ensure some network resilience in unforeseen circumstances (pipe failure, large fire flows, etc).

The surplus power factor s could also be applied to analyze the hydraulic reliability of a WDS. If a WDN model is available, the s factor calculation is straightforward and does not require more computing power than a usual hydraulic model calculation procedure.

The analysis of the surplus power factor s and the coefficient of network efficiency η_n could be applied to reconstruct existing or design new WDNs. The case study for a simplified optimization procedure (section 4.2.5) revealed that by increasing or decreasing pipe diameters or pressure for pumps will affect the s

value. It shows that the surplus power factor and the coefficient of network efficiency are directly related to the head loss developed in the system.

If the *s* factor is reaching its minimum value (s=0) and the flows are still increased, the s value would start to increase again, giving wrong results. Therefore, the coefficient of the network efficiency η_n is used to validate the calculation of the *s* factor in the system. If η_n<2/3, then *s* has reached its minimum value. Comparison of graphical results of both coefficients enables us to find out any discrepancies that could occur in the analysis of complex WDNs.

The cost for WDNs has always been a factor for the development of different types of optimization models. Although the cost of optimization is not the topic of this thesis, one commercial software package was tested in order to compare the results of *s* value to the alteration of the system cost. The results show that readily available software packages can be used in order to link the system cost with resilience.

Focus in the second case study was resilience of the Viimsi WTS. The hydraulic model is relatively small consisting of 10 pipes between the source and target nodes. The exact data for pump curves and pipe diameters allowed to linking the network and pump efficiency and calculation of the overall system efficiency η_s. Since the hydraulic model is steady state and unpredictable conditions (fire flows, demand variations, etc) are excluded, the system resilience is rather low, *s* being in the range of 6% to 24%. It should be considered acceptable since the system draws groundwater from two different water tables, with each level supplying the treatment plant with its own pipeline. Also, the location of the clean water reservoir works as a water tower in case of major electrical malfunction in the area. The interaction between the water network and the pumps revealed that the efficiencies of the pumps and network need to be investigated in joint operation.

The research conducted during the preparation of this thesis shows that it is not sufficient to take into consideration only everyday diurnal consumption. Pipes in WDNs have a lifespan of at least 50 years, sometimes even more. It is difficult to forecast the future and design WDNs that meet today's and tomorrow's needs since new technologies in the water industry and economic reasons plus the development of cities (usually growing together with nearby settlements) tend to be quite unpredictable. If the parts of WDNs in the cities start to deteriorate (depending on piping age or maintenance quality), the main question for rehabilitation is whether the diameters for renewed pipes should be decreased, increased or left as they were. For the last decade, the common practice in Tallinn was to decrease renewed pipe diameters significantly, since the water consumption decreased. On the other hand, resulting from higher economic welfare, in the newly developed suburban residential areas, Tallinn WDNs have been growing. Now the decisions (decreasing pipe diameters) made 5 to 10 years ago challenge the growth of the system and some bottlenecks have already been unfolded. However, more practicable decisions could have been made if along with the studies focused on quality, some sort of capacity analysis had been made. The approach introduced in this thesis enables us to achieve better results in the analysis of WDN reliability against unforeseen demand conditions.

5.2. Recommendations for further research

Based on the study conducted on the Õismäe-Mustamäe WDN, a minimum *s* value of 20% was recommended. For solid conclusions, different types and scales of WDNs should be analyzed. Also, the minimum fire flow requirements can differ country-by-country, which also affect the recommended *s* value.

The main area to be further developed is the joint operation between the pumps and WDN. The Õismäe-Mustamäe hydraulic model used in the analysis was calibrated, but the data for pumps were inadequate in order to develop an approach for analysing the pump-network interaction. Therefore the approach presented in sections (2.5) and (2.6) has only theoretical value that is not tested on in-service WDSs.

As Wu et al. (2011) showed, the resilience concept could be applied to the optimization procedures that consider multi-objective trade-off between the cost and resilience indexes. In this thesis readily available WaterGEMS software was used in order to compare the cost and resilience of the Õismäe-Mustamäe WDN. However, this area needs further development in order to connect the resilience and cost aspect directly to each other without the interference of interim software steps - in this case WaterGEMS' Total Benefit feature.

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ABSTRACT

This thesis studies the aspects of hydraulic power capacity of water distribution systems (WDS). The thesis gives a historical overview of different reliability and resilience indexes that have been applied to WDS analysis whether to increase the hydraulic reliability of the system or to apply the indexes for cost optimisation purposes.

In the research anew resilience index called surplus power factor *s* and network efficiency coefficient η_n were developed. The factor *s* characterizes the reliability of the hydraulic system. The value will vary between 1 and 0. If s=0, the hydraulic system works at a maximum capacity. The increase of the value of the factor *s* will improve system reliability until it reaches its maximum value 1. The hydraulic power equations were derived to prove the validity of the surplus power factor *s* for any water distribution network (WDN), not only for a single transmission pipeline. The coefficient η_n enables one to determine the efficiency of WDN as well as to validate the calculation of the *s* factor.

Two case studies were conducted. Factor *s* and network efficiency coefficient η_n were determined for in-service WDNs in different demand (maximum, average and minimum) and fire flow conditions. The study showed that the *s* factor and η_n are directly related to the head loss developed in the system, which in turn is related to the water distribution system parameters - WDN topology (i.e., pipe diameter, roughness, pump station location) and demand conditions. The results showed that for an average demand condition the *s* value is around 0.6. If the fire flows are applied during maximum demand conditions, the *s* factor decreases significantly, reaching the values close to 0.1. This indicates that the system has some reserve left in order to withstand larger than everyday normal demands (i.e. fire flows). According to the case study, the minimum *s* value for WDNs proposed is 0.2...0.3 in maximum demand conditions, which allows the WDS to cope with additional fire demands or future extensions.

In order to calculate the *s* factor for WDNs, a network resistance coefficient *C* has to be determined. The coefficient *C* characterizes the overall head losses in water pipelines and is the basis for the *s* factor calculation. In this thesis a theoretical approach for determining the coefficient *C* through matrix equations is presented. In practice, the method uses EPANET software, which does the hydraulic simulation of a WDN and theoretical equations in order to calculate the *C* value for a WDN based on head losses in each pipe.

KOKKUVÕTE

Käesolevas doktoritöös uuritakse veevõrkude hüdraulilise võimsuse ülekannet. Doktoritöö annab ajaloolise ülevaate veevõrkude töökindluse uuringutest, mida on rakendatud süsteemi töökindluse suurendamiseks või maksumuse optimeerimise hindamiseks.

Teadustöö tulemusel on välja töötatud uus hüdrauliliste võrkude iseloomustamise varutegur s, mille väärtus varieerub piirides nullist üheni. Kui s=0, siis süsteem töötab oma hüdraulilise võimsuse maksimumil. Varuteguri väärtuse suurenemine tõstab hüdraulilisest seisukohast süsteemi töökindlust kuni see saavutab oma maksimumväärtuse (s=1). Hüdraulilise võimsuse ülekande võrrandid on tuletatud varuteguri arvutamiseks üksikus torus ning erinevat tüüpi veevõrkude süsteemides. Koefitsient η_n võimaldab hinnata veevõrkude efektiivsust ning kontrollida s teguri arvutust.

Doktoritöös viidi läbi kahe erinevat tüüpi veevõrgu arvutusanalüüs. Varutegur s ja veevõrgu efektiivsus η_n määrati erinevate tarbimisrežiimide, sealhulgas tulekahju olukorras. Analüüs näitas, et varutegur s ja efektiivsus η_n on otseses seoses rõhukadudega veevõrgus, mis omakorda on seotud veevõrgu topoloogiaga (torude diameetrid, karedused pumpla asukoht) ja tarbimisrežiimiga. Tulemused näitavad, et keskmise tarbimisrežiimi korral on s väärtus veevõrgus 0,6. Tulekahju olukorras ja samaaegselt maksimaalse tarbimisrežiimi korral väheneb varutegur s märkimisväärselt kuni väärtuseni 0,1. Sellest järeldub, et veevõrgus on reserv, mis võimaldab tulla toime suuremate tarbimistega kui igapäevane tavaline tarbimisrežiim (näiteks tulekahju olukord). Tuginedes analüüsile on soovituslik varutegur maksimaalse tarbimise korral veevõrkudele 0,2...0,3, mis võimaldab tulevikus toime tulla veevõrgu perspektiivsete laiendustega või tulekahju olukordadega.

Varuteguri s määramiseks on vaja eelnevalt arvutada veevõrgu takistustegur C, mis iseloomustab rõhukadusid kõikides torudes. Doktoritöös on kirjeldatud teoreetiline meetod C arvutamiseks kasutades maatriksvõrrandeid. Praktilise meetodina kasutatakse EPANETi tarkvara, mis teostab veevõrgu hüdraulilise simulatsiooni ning teoreetilisi valemeid millega arvutatakse C väärtus kogu veevõrgule.

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2000-2007	ETP Grupp AS	Osakonnajuhataja, insener
1999-2000	Eesti Tööstusprojekt RAS	Insener

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Õismäe ja Mustamäe linnaosade veevõrgu hüdraulilise ning vee kvaliteedi arvutusmudeli koostamine, bakalaureusetöö

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APPENDIX A: Demand Patterns for the Õismäe-Mustamäe WDN

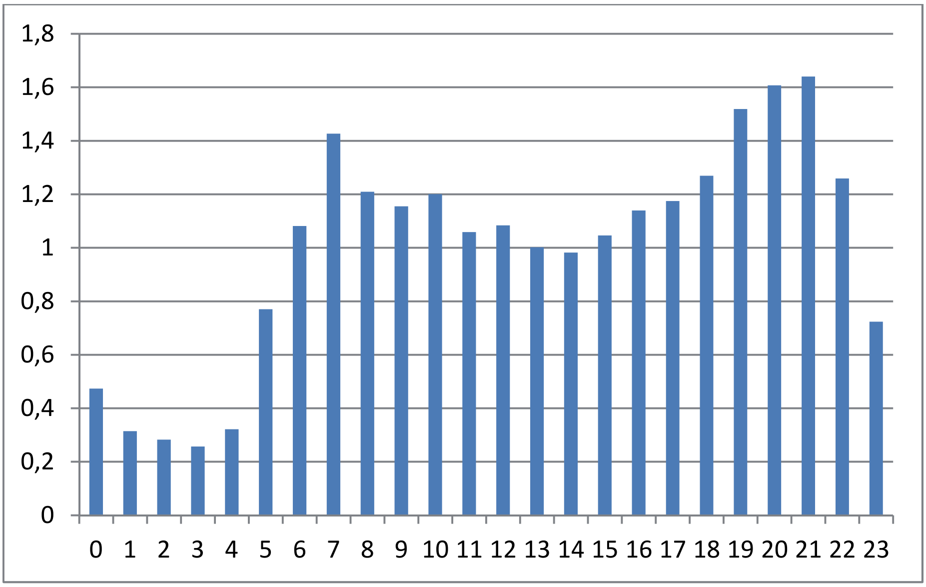

Figure A. 1 Demand pattern for small houses

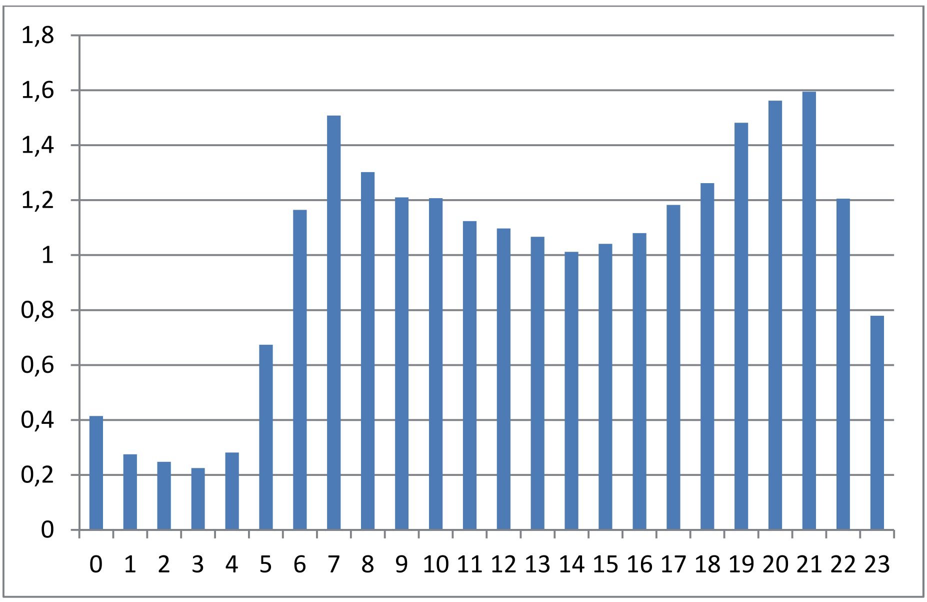

Figure A. 2 Demand pattern for large buildings

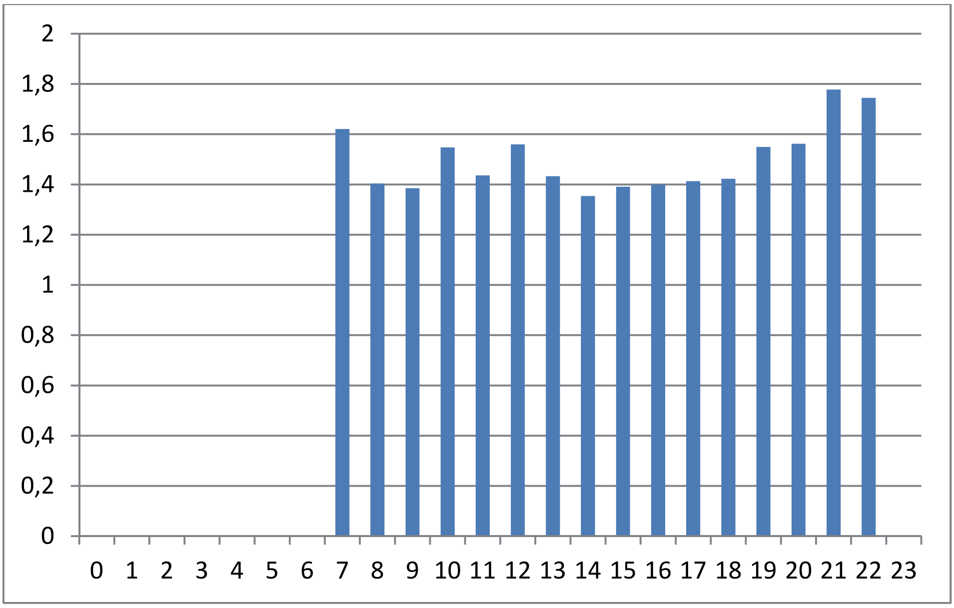

Figure A. 3 Demand pattern for a large industrial building

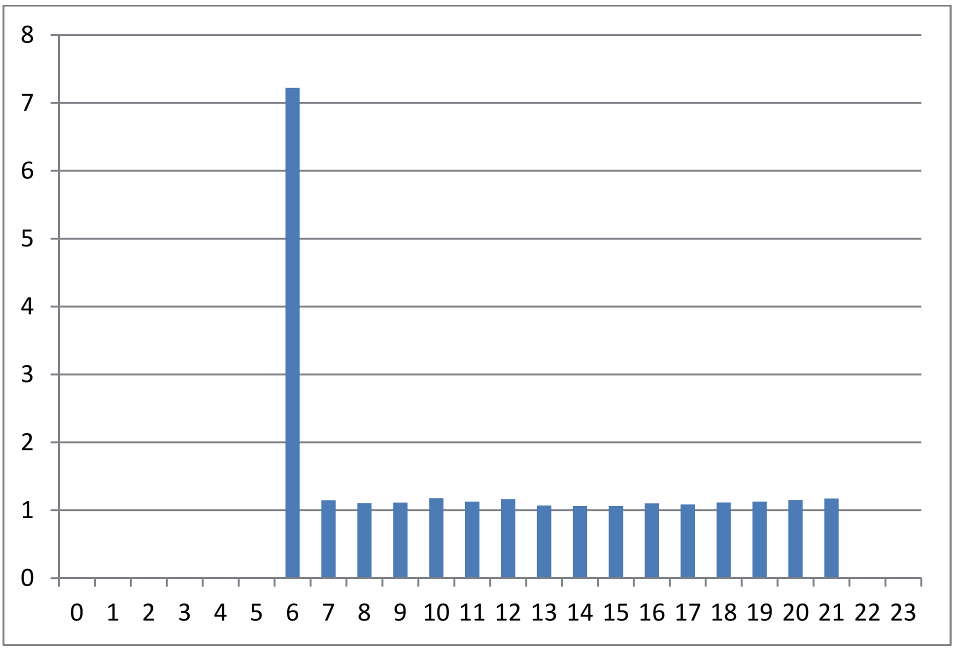

Figure A. 4 Demand pattern for a large social building

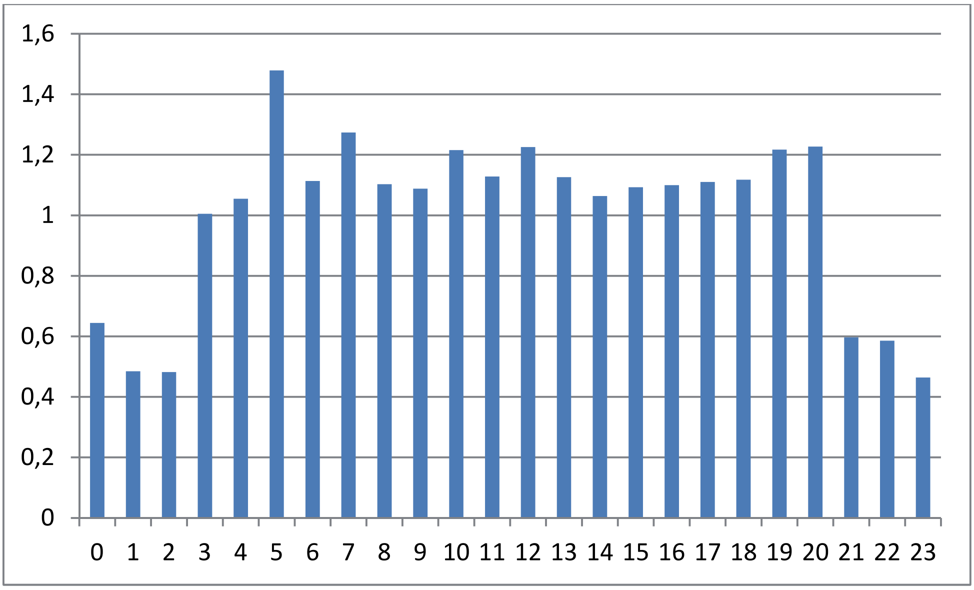

Figure A. 5 Demand pattern for major buildings

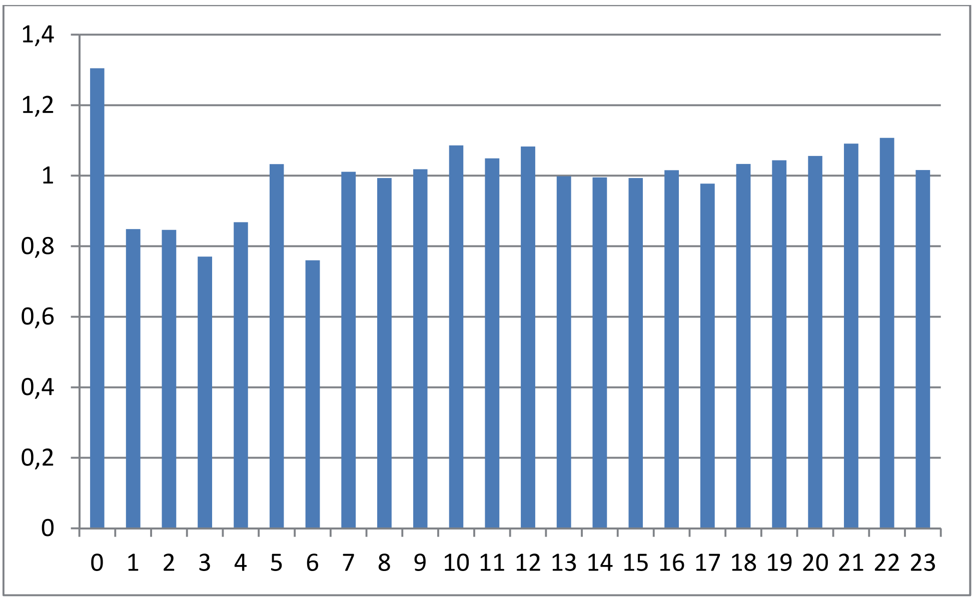

Figure A. 6 Demand pattern for leakages

DISSERTATIONS DEFENDED AT TALLINN UNIVERSITY OF TECHNOLOGY ON *CIVIL ENGINEERING*

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