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Capacity reliability of water distribution systems

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ABSTRACT

Hydraulic power capacity of the water distribution network (WDN) is analyzed, and energetically maximum flows in pipes and networks are determined. The concept of hydraulic power for the analysis of WDN characteristics is presented. Hydraulic power capacity characterizes the WDN capacity to meet pressure and flow demands. A capacity reliability indicator called the surplus power factor is introduced for individual transmission pipes and for distribution networks. The surplus power factor s that characterizes the reliability of the hydraulic system can be used along with other measures developed to quantify the hydraulic reliability of water networks. The coefficient of the hydraulic efficiency η_n of the network is defined. A water distribution system in service is analyzed to demonstrate the s and η_n values in the water network in service under different demand conditions. In order to calculate the s factor for WDNs, a network resistance coefficient C was determined. The coefficient C characterizes overall head losses in water pipelines and is a basis for the s factor calculation. This paper presents a theoretical approach to determine the coefficient C through matrix equations.

Key words | hydraulic efficiency, hydraulic power, surplus power factor, water distribution network

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INTRODUCTION

Resulting from the structural changes in society, water consumption regimes changed essentially after independence was restored in Estonia in 1991. Lower water consumption has led to oversized urban water networks. For example, in Tallinn, overall water consumption decreased over two times during the first 5 years of independence. The main reasons can be summarized as follows: leakage reduction, influence of the increased water price on the control of consumption (practically all home consumers installed a water meter), and reduced industrial consumption. Water distribution networks (WDNs) had been constructed according to the Soviet Standard SNIP for water consumption rates which had a drastic influence on the quality of water in the network. Flow velocities were very low and the calculated retention time of water in the network before consumption was high. Therefore, extensive rehabilitation programs for WDNs were applied in 1996. The main criterion for WDN reconstruction was hydraulic reliability, i.e., to guarantee that good quality water is delivered to consumers at any time under sufficient pressure. Water age used to forecast changes in water quality was considered in the WDN rehabilitation strategy development. The aim of this article is to introduce new hydraulic capacity reliability indicators to the reconstruction process of WDNs: the surplus power factor s and the coefficient of hydraulic efficiency η_n of the network.

The concept of hydraulic power developed by Park *et al.* (1998) was used to assess the hydraulic reliability of a WDN. The performance of a WDN depends on the ability of the network to meet the demands of volume and pressure of the flow. To evaluate the network simultaneously on the basis of the flow and the pressure, the concepts of hydraulic power and energy transmission can be applied. In this research, two main characteristic parameters 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

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allows us to look at a feasible flow of hydraulic power in a single pipe or in the WDNs and to analyze the hydraulic reliability of the system (Park *et al.* 1998).

Schneiter *et al.* (1996) used the concept of hydraulic power capacity to identify the pipes subject to rehabilitation which would mostly contribute to increase the hydraulic capacity of the network. Their analysis of rehabilitation is based on a capacity-versus-cost tradeoff curve, i.e., the volume flow was used singly for their rehabilitation estimates.

Todini (2000) used the concept of hydraulic power to analyze the resilience index that characterizes the surplus of energy capability to overcome sudden failures in looped networks. If more power than required is delivered to WDN consumption nodes, a surplus is available for dissipation in the network in case of failures (Todini 2000). Based on the concept of resilience, Prasad & Park (2004) introduced a new resilience measure called 'network resilience'. Network resilience calculations take into account the effects of both surplus power and reliable loops. Blackmore & Plant (2008) used the resilience theory with integrated urban water systems. Thus, risk management and resilience concepts were applied in urban water systems.

Energetically optimal flows in an individual pipe and in a series of pipelines with discrete and continuous distribution of water consumption with time variability have been discussed by Ainola *et al.* (2003). Energetically optimal head distribution in pipes were determined, and the results have helped to increase the efficiency of energy consumption by the rehabilitation of WDNs.

The coefficient of the critical outlet power k and the surplus power factor s were introduced by Vaabel *et al.* (2006) to evaluate the hydraulic power capacity of a water distribution system (WDS) on the basis of both flow within pipes and pressure head at the inlets of pipes. The value of the critical outlet power k characterizes the potential of the hydraulic power used by the hydraulic system. At the same time, it enables us to determine the reserve of hydraulic power. For the latter, the surplus power factor s was defined. This factor characterizes the reliability of the hydraulic system. If $s = 0$, then the hydraulic system works at a maximum capacity. The increase of the value of s will improve the hydraulic reliability of the system.

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) have also compared the surplus power factor with three commonly used network resilience measures: the resilience index (Todini 2000), the minimum surplus head (Gessler & Walski 1985), and the modified resilience index (Jayaram & 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). The surplus power factor has an advantage over existing network resilience measures. In the calculation of the surplus power factor, the value of the output pressure head of the network is not required, 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).

Ostfeld *et al.* (2002) have used stochastic simulation for the reliability analysis of a WDS, taking into account the quality of the water supplied as well as hydraulic reliability considerations. Their analysis was carried out for multi-quality water distribution systems.

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 was 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 coefficient of the hydraulic efficiency η_n of a network is defined. A water distribution system in service was analyzed to demonstrate the suitability of s and η_n values in a real water network under different demand conditions.

HYDRAULIC ENERGY TRANSMISSION IN A PIPE

Hydraulic power capacity shows that the WDN capacity meets pressure and flow demands. It is a measure of system reliability defined as a probability of a feasible flow of hydraulic power existing in the pipe or in the network.

Hydraulic power in a simple case in an individual pipe was examined by Vaabel *et al.* (2006). 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 1). Flow and head units are represented in liters per second and in meters correspondingly.

The following equations are valid for the steady-state conditions.

We have for heads

$$H_0 = h + H_1 \tag{1}$$

In general, head loss can be expressed as

$$h = cq^a \tag{2}$$

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

For an individual pipe

$$Q_1 = Q_0, q = Q_0, h = cQ_0^a \tag{3}$$

The hydraulic powers in a pipe are defined as

$$P_0 = \gamma Q_0 H_0, P_d = \gamma c Q_0^{a+1}, P_u = \gamma Q_0 H_1 \tag{4}$$

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.

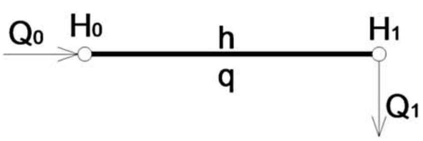

Figure 1 | Pipe with flows, heads, and head loss.

We can write

$$P_u = P_0 - P_d \tag{5}$$

or

$$P_u = \gamma(Q_0 H_0 - cQ_0^{a+1}). \tag{6}$$

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 Equation (6) under the condition $dP_u/dQ_0 = 0$ it follows that

$$H_0 - (a+1)cQ_{0\,\max}^a = 0 \tag{7}$$

Now, from Equations (6) and (7) we obtain

$$P_{u\,\max} = \gamma c a Q_{0\,\max}^{a+1} \tag{8}$$

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

$$k = \frac{P_u}{P_{u\,\max}} \tag{9}$$

Using Equations (6)–(8), the coefficient k through the flows can be expressed as

$$k = \frac{a+1}{a}\left[1 - \frac{1}{a+1}\frac{Q_0^a}{Q_{0\,\max}^a}\right]\frac{Q_0}{Q_{0\,\max}} \tag{10}$$

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

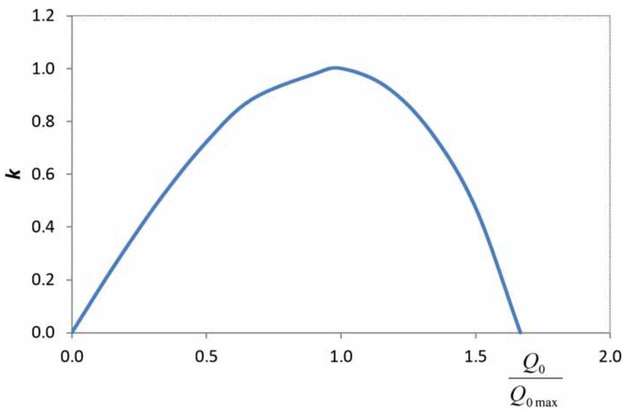

Figure 2 | Coefficient of the critical outlet power k as the function of the ratio $Q_0/Q_{0\max}$.

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

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 3)

$$s = 1 - k \tag{11}$$

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. An increase of the value of the factor s will indicate improvement of system reliability until it reaches its desirable value ($s = 0.2 \ldots 0.3$).

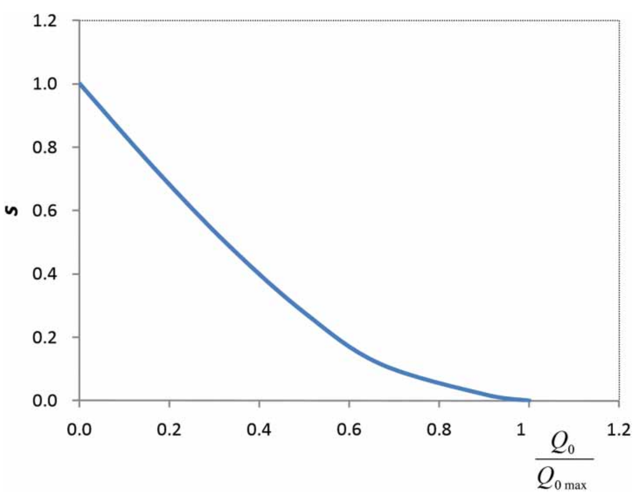

Figure 3 | Comparison of surplus power factor to the flow, $a = 2$.

The considerations given here for an individual pipe can be generalized for any WDN.

Theoretically Q_0 could be greater than $Q_{0\,\max}$, but it is not practical since very high input power values are required to achieve a certain s value, which results in extremely low efficiency within the system. Therefore, it could be ignored in the network resilience estimation of a WDS (Wu *et al.* 2011).

HYDRAULIC EFFICIENCY OF THE WDNS

Next let us examine the hydraulic power in a WDN.

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:

$$p = n + l \tag{12}$$

Assume that the network topology is given by the following incidence matrices: first, the unknown head nodes incidence ($p \times n$) matrix (Rossman 2000):

$$A = [a_{ij}] \text{ where } a_{ij} = \begin{cases} 1 & \text{if the flow of pipe } i \text{ enters node } j, \\ 0 & \text{if pipe } i \text{ is not connected with node } j, \\ -1 & \text{if the flow of pipe } i \text{ leaves node } j. \end{cases} \tag{13}$$

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

Second, the fixed head node incidence ($p \times 1$) matrix:

$$B = [b_i] \text{ where } b_i = \begin{cases} -1 & \text{if the flow of pipe } i \text{ comes from the fixed head node,} \\ 0 & \text{if pipe } i \text{ is not connected with the fixed head node.} \end{cases} \tag{14}$$

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

$$Q = \begin{pmatrix} Q_1 \\ Q_2 \\ \cdots \\ Q_n \end{pmatrix}, q = \begin{pmatrix} q_1 \\ q_2 \\ \cdots \\ q_p \end{pmatrix}, H = \begin{pmatrix} H_1 \\ H_2 \\ \cdots \\ H_n \end{pmatrix} \tag{15}$$

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

The head loss $(p \times 1)$ vector h can be expressed as

$$h = Dq \tag{16}$$

where D is the hydraulic impedance matrix in the form

$$D = \begin{bmatrix} c_1|q_1|^{a-1} & 0 & \dots & 0 \\ 0 & c_2|q_2|^{a-1} & \dots & 0 \\ \dots & \dots & \dots & \dots \\ 0 & \dots & 0 & c_p|q_p|^{a-1} \end{bmatrix} \tag{17}$$

The unknown head H_i and the flow vector components q_k are determined from the following energy and mass conservation laws (Ormsbee & Wood 1986):

$$AH + Dq = -BH_0, A^T q = Q, B^T q = Q_0 \tag{18}$$

where A^T and B^T are the transpose of matrices A and B. Let us note that in Equation (18) 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 (1×1) – equation.

Here the continuity of the flow rate

$$Q_0 = \sum_{i=1}^{n} Q_i \tag{19}$$

holds. Here Q_i is the flow into pipe i.

The network characteristic is expressed as

$$H_0 = F(Q_0) \tag{20}$$

where the function F is determined through the system of Equations (18) and (19). 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

$$\tilde{Q} = \frac{Q}{Q_0}, \tilde{q} = \frac{q}{Q_0}, \tilde{H} = \frac{H}{Q_0^a}, \tilde{D} = \frac{D}{Q_0^{a-1}} \tag{21}$$

where $\tilde{Q}$, $\tilde{q}$, $\tilde{H}$, and $\tilde{D}$ are variable correction matrices for tentative values of nodal demands, flows, heads, and the hydraulic impedance matrix. In fact, in Equation (21) we are normalizing all our constituents regarding Q_0. 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, Equations (18) and (19) can be written in the form

$$(A\tilde{H} + \tilde{D}\tilde{q})Q_0^a = -BH_0 \tag{22}$$

and

$$A^T\tilde{q} = \tilde{Q}, \ -B^T\tilde{q} = 1, \ \sum_{i=1}^{n} \tilde{Q}_i = 1. \tag{23}$$

By multiplying both sides with B^T and taking into consideration that $B^T B = \beta$, we obtain from Equation (22)

$$H_0 = -\frac{1}{\beta} B^T (A\tilde{H} + \tilde{D}\tilde{q})Q_0^a \tag{24}$$

where β is the number of units among the matrix B elements. Let us note that the dimension of Equation (24) is (1×1).

With the denotation

$$C_{\text{tot}} = -\frac{1}{\beta} B^T (A\tilde{H} + \tilde{D}\tilde{q}) \tag{25}$$

Equation (24) takes the form

$$H_0 = C_{\text{tot}} Q_0^a. \tag{26}$$

Here C_{tot} is the resistance coefficient of the water network that characterizes the condition when the hydraulic power has been dissipated completely in the system. In general, it depends on $\tilde{q}$ and $\tilde{D}$ elements and on the head in one point of the network. Equation (26) characterizes the condition when head loss in the system equals the 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 Equation (1). This provides necessary pressure in consumer nodes. From Equation (25) the dimension of C_{tot} is (1×1). The values of $\tilde{H}$, $\tilde{D}$, and $\tilde{Q}$ are changing in time, but the value of C_{tot} from Equation (26) is constant.

The hydraulic power of the water network can be expressed as

$$P_0 = \gamma Q_0 H_0, P_d = \gamma (Dq)^T q, P_u = \gamma [Q_0 H_0 - (Dq)^T q] \quad (27)$$

and

$$(D_q)^T q = (\tilde{D} Q_0^{a-1} Q_0 \tilde{q})^T Q_0 \tilde{q} = Q_0^{a+1} \tilde{q}^T \tilde{D} \tilde{q} \quad (28)$$

From Equations (16) and (27) it follows that

$$C = \tilde{q}^T \tilde{D} \tilde{q}, \quad (29)$$

where C is the resistance coefficient of the WDN.

In practice, network equations can be solved with appropriate hydraulic software. We used EPANET (Rossman 2000), which employs the gradient method (Todini & Pilati 1988) to solve matrix equations.

Based on Equation (29), for $a = 2$, C can also be expressed as

$$C = \frac{\sum_i (h_i \cdot q_i)}{Q_0^3} \quad (30)$$

where h_i is the head loss in pipe i, and q_i is the 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 Equation (30).

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

$$\eta_n = \frac{P_u}{P_0} \quad (31)$$

we have

$$P_u = \eta_n P_0 \text{ or } P_u = \gamma \eta_n H_0 Q_0. \quad (32)$$

From Equations (27) and (31) we obtain

$$\eta_n = 1 - \frac{C Q_0^a}{H_0} \quad (33)$$

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 Equation (32) under the condition

$$\frac{dP_u}{dQ_0} = 0 \quad (34)$$

it follows that

$$\frac{d\eta_n}{dQ_0} Q_0 + \eta_n = 0 \quad (35)$$

Substituting Equation (33) into Equation (35) we obtain

$$Q_{0\,\max} = \left[\frac{H_0}{(1+a)C} \right]^{1/a} \quad (36)$$

Equation (36) for the network formally coincides with Equation (7) for an individual pipe. Therefore, Equations (8) to (10) and (11) are applied also to the WDN as a whole.

Now, from Equations (33) and (36) it follows

$$\eta_n = \frac{a}{1+a} \quad (37)$$

Thus, for any H_0 the hydraulic efficiency of the network η_n at the maximum value of P_0 is constant, given by Equation (37) (Figure 4).

CASE STUDY

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, Estonia. The case study used an approach where η_n and s values are determined at the location of the network node. It is evident that the farther the node from the pumping station, the larger is the pressure loss in the system. Therefore, the surplus power factor is smaller than in the nodes closer to the pumping station.

The case study covered the Õismäe–Mustamäe area. The total length of pipelines in this pressure zone is

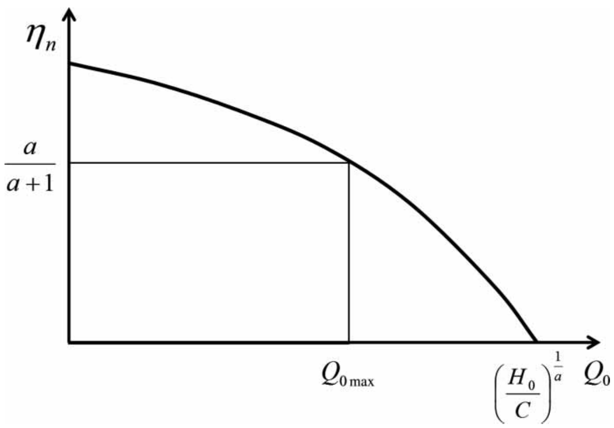

Figure 4 | Hydraulic efficiency of the network as the function of Q_0.

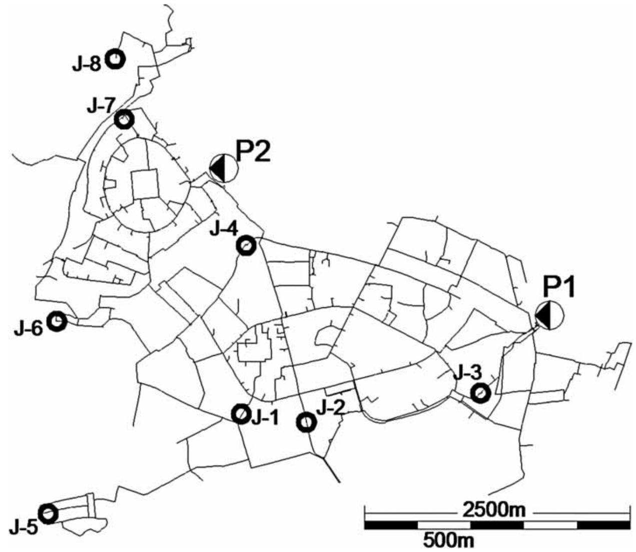

Figure 5 | Layout of the Õismäe–Mustamäe hydraulic model with fire flow locations.

approximately 85 km. Since all consumers are equipped with flow meters, the water company has a 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, although 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 system is supplied via two booster pumping stations: P1 and P2 (Figure 5). 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 night time, the pressure in the pumping station P1 is lowered. As Figure 5 shows, choice of the location of the main source (P1) for energy is not well-grounded to ensure an even distribution of power, although it is compensated by a higher ground level than most of the nodes in the network.

In our approach, the network efficiency η_n and the surplus power factor s were calculated for daily maximum, minimum, and average demand conditions as well as for maximum demand condition in a fire flow situation. Fire flows were selected in the range of 10 to 15 L/s according to the area's characteristics (private houses, apartment buildings, etc.). 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 WDN model itself consists of 2,480 nodes and 2,560 pipes. The analysis was completed on the calibrated WDN with the EPANET software at three different time steps and fire flow conditions, as described above. In the hydraulic model definition, leakages were applied into the demand nodes. Therefore, the calculation for the s factor also took into account the leakages in the WDN.

First, the analysis results for all pipes (flow, length, diameter, and friction factor) were exported and head loss in each pipe was calculated. Based on Equation (30), the resistance coefficient of the water network was calculated. In order to calculate the coefficient of the critical outlet power k, first, $Q_{0\max}$ was determined by accounting the total head at the pumping station (Equation (36)). After that k was determined according to Equation (10) with the flow exponent $a = 2$. The surplus power factor was then calculated by Equation (11) and the network efficiency η_n according to Equation (31). If $\eta_n = 2/3$ or less, the calculation of k reaches above its optimum point $k = 1$ and starts to decrease again (i.e., s will increase), as seen in Figure 2. Therefore, if $\eta_n = 2/3$ or less, the calculation results of k could be ignored and $s = 0$. Another parameter can also be used to confirm the correct calculation of s value. If $Q_0 > Q_{0\max}$, then s has also reached its minimum value.

The above-mentioned steps were performed for different demand conditions and with all the nodes J-1 to J-8 under fire flow conditions.

RESULTS

Tables 1 and 2 show the results for network efficiency η_n and surplus power factor s calculations in the Õismäe–Mustamäe WDN.

Table 1 | Surplus power factor calculation results at different time steps

	6:00 am	4:00 pm	2:00 am
C	68.7	40.9	174.2
Q_0, L/s	285.38	198.37	58.51
H_0, m	71.64	71.53	60.24
$Q_{0\max}$, L/s	589.6	763.5	339.5
s	33.1%	61.9%	74.4%
η_n	92.2%	97.7%	99.0%

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

As shown, the farthest location has the lowest possibility of serving a large demand capability compared to the node 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 closer to the source node, for example, nodes J-3 and J-8, since the head loss between the source and the target nodes is increased significantly.

The results for η_n and s values are plotted in Figures 6 and 7. At all conditions, η_n values are higher than 2/3 (Figure 6). If $\eta_n = 2/3$, the surplus power factor s reaches its minimum value ($s = 0$) and has no capacity for further decrease. Therefore, it is required to calculate η_n values first and only then the critical outlet power k and the surplus power factor s can be determined. If η_n value is omitted and s is calculated directly according to Equations (10) and (11), the results obtained could be wrong since the minimum value of s could already be exceeded.

As could be seen from the results, a good target value for s depends significantly on local fire-fighting regulations and on 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 they decrease significantly depending on the fire flow demand and location in the WDN. To propose certain s values for a new design or rehabilitation of existing WDSs, different types of WDS should be studied. At maximum

Table 2 | Surplus power factor calculation results at 6:00 am + fire flow at 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	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
Fire flow, 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, m	66.14	66.14	66.14	66.14	70.02	66.14	66.14	66.14
$Q_{0\max}$, L/s	539.7	554.2	594.5	520.6	543.7	529.6	529.6	423.3
s	25.1%	26.7%	30.7%	23.1%	26.5%	24.0%	24.0%	11.4%
η_n	89.7%	90.2%	91.5%	88.9%	90.2%	89.3%	89.3%	83.2%

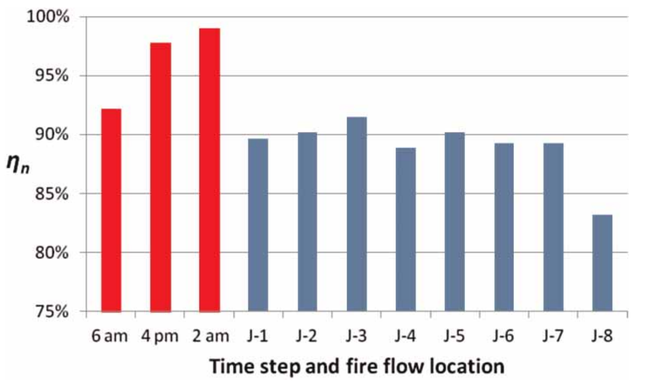

Figure 6 | Hydraulic efficiency of the network at a given time and given fire flow location (6.00 am).

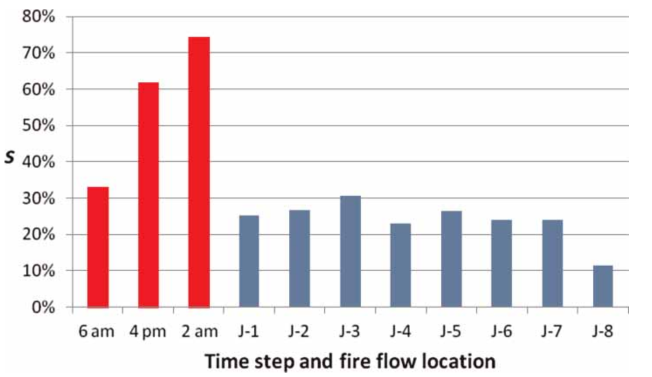

Figure 7 | Surplus power factor at network nodes at a given time and given fire flow location (6:00 am).

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.

CONCLUSIONS

The performance of a WDN depends on the ability of the network to meet the demands for the flow volume and pressure. 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 system, it is sometimes complicated to apply their findings into everyday practice.

Focus in this paper is on the development of the surplus power factor s and the coefficient of the network efficiency η_n. As a novel feature, a new integral parameter in the reliability analysis of the WDN hydraulic capacity is introduced. 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 maximum capacity. The increase of the factor s value indicates improvement of system reliability until it reaches its desirable value ($s = 0.2 \ldots 0.3$). The hydraulic power equations were derived to prove the validity of the surplus power factor s for any WDN, not only for a single transmission pipeline. In a broader aspect, the analysis of the surplus power factor could be applied to achieve higher operational efficiency of a WDS when calculating the efficiency of a pumping station.

Within a case study for an existing WDN (2,480 nodes and 2,560 pipes), an analysis was conducted for different time steps (maximum, minimum, and average demand conditions) and fire flow conditions. The study showed that in average demand conditions, η_n and s values are around 97 and 62%, while under fire flow conditions (with maximum diurnal demand) a decrease to 83 and 11%, respectively, occurred.

The surplus power factor is not required to calculate 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). In the WDN surplus power factor analysis, existing software packages, e.g., EPANET, could be used. However, it is complicated to apply the analysis for WDSs with several water sources or pressure zones. The analysis for large WDSs with multiple pressure zones should be conducted on each pressure zone separately.

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 requires no 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 study has shown that the surplus power factor and the coefficient of network efficiency are directly related to the head loss developed in the system, which in turn is related to the WDS parameters – WDN topology (i.e., pipe diameter, roughness, pump station location) and demand conditions.

Calculation of s values is quite straightforward, although some details should not be ignored. 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 $\eta_n < 2/3$, then s has reached its minimum value. Comparison of graphical results of both coefficients enables us to discover any discrepancies that could occur in the analysis of complex WDNs.

The case study has shown that the theory is well applicable to the analysis of the WDN performance under different demand conditions. Initially, it may seem that water networks are quite over-dimensioned, considering a normal diurnal demand. Therefore, the surplus power factor for average demand conditions is around 60%. However, if the fire flows are applied during maximum demand conditions, the surplus power factor decreases significantly, reaching values close to 10%.

In order to propose certain s values for urban networks, different models should be analyzed. The new parameters s and η_n proposed in the article could be used in new WDN design or old WDN rehabilitation process. If parts of urban WDNs start to deteriorate (depending on piping age or maintenance quality), the key issue for rehabilitation is whether the diameters for renewed pipes should be decreased, increased, or left as they are. The approach introduced here enables us to achieve better results in the analysis of WDN hydraulic reliability.

It was not within the scope to analyze different leakage values in this paper. However, since leakages may affect the results of the analysis it would be advisable to apply different leakage values into the hydraulic models and estimate the s factor sensitivity against it.

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