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Complex approach to the water network model calibration and the leakage distribution

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Abstract

The distribution of water losses due to leakage from the water network is studied as the first step of the mathematical model calibration procedure. In order to calculate the leakage from pipe an empirical formula which takes into account the length of pipe and mean pressure, and also contains certain coefficients, is used. This leakage is carried to the terminal nodes and distributed between the nodes proportionally to the pressure in the nodes. For determination of leakage the measured pressure in control nodes is used. For evaluation of pressure in other nodes an iteration process is applied which determines the leakage and pressures intermittently. Computational results for water distribution network of the city centre of Tallinn are presented, demonstrating the applicability of the method.

1 Introduction

In the course of calibration of the simulated model of water network the main parameters which can be modified during the adjustment phase are uncontrolled consumptions and pipe roughnesses. If the values of the obtained roughnesses are not acceptable, effective diameters of the pipe can be changed. The uncontrolled consumptions are due to leaks, illegal connections, meter errors, etc. It is very difficult to estimate these volumes and their spatial and time distribution. Only the total volume of uncontrolled consumptions can be easily determined evaluating the difference between the volume of total input and the

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volume of registered water. The volume of registered water over the input volume of a certain period of time is called the average efficiency of the network. In the case of high average efficiency the spatial distribution of uncontrolled consumption between the model nodes is less important than in the case of low average efficiency. If the average efficiency is around 85 – 90 %, the traditional method, which adopts the hypothesis of uniform efficiency throughout the network, can be used (Martinez & Garcia-Serra [1]). This procedure assumes proportionality between the controlled and uncontrolled flow rates in the nodes. If the average efficiency is smaller, the unproportionality in the distribution of uncontrolled consumptions must be taken into account. Since it is difficult to determine the components of uncontrolled consumption, the whole uncontrolled flow is usually treated as leakage. Some methods are proposed for evaluation of the distribution of leaks in the network. Vela et al. [2] proposed to simulate a leak with a fictitious discharge valve for this purpose and to determine the number of defects by statistical criteria and historical data. Germanopoulus [3] gives a formula for determination of leakage losses from the network as

$$S_{ij} = c_1 L_{ij} \left(p_{ij}^{av}\right)^{1.18}, \tag{1}$$

where S_{ij} is the leakage outflow from the pipe connecting nodes i and j, which is distributed equally between nodes i and j, c_1 is a constant depending on the characteristics of the particular network, L_{ij} is the pipe length, and p_{ij}^{av} is the average pressure along the pipe. This formula is based on an empirical expression obtained earlier by other authors from experimental results of the ratio of leakage index and average zone night pressure. Tucciarelly et. al. [4] used in their simulation model for water losses the formula

$$Q_i = \left(H_i - z_i\right)^a \sum_{j=1}^{M_i} \frac{\pi}{2} D_{ij} \theta_{ij} L_{ij}, \tag{2}$$

where Q_i is the water loss per time unit from small leaks close to node i, H_i is the total water head, z_i is the topographic elevation, a is a loss exponent, D_{ij} is the pipe diameter, θ_{ij} is the leak's surface per unit pipe surface of the pipes linking nodes i and j, and M_i is the total number of pipes linked to node i. The loss factors θ_{ij} together with the roughness factors are proposed to be determined in the calibration process of the network. With reference to decreasing the number of variable parameters it is assumed that the loss factors and roughness factors are homogeneous within the specified areas.

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2 Distribution of the leakages

In order to begin with the calibration procedure of the water network model, the magnitude and distribution of water losses due to leakage must be considered. We proceed from the calculation of the network model with the assumption that leakages are proportional to the controlled flow rates. Regarding improvement the obtained distribution of leakage between the nodes let us take into account some information about the pipes and measured pressure in certain nodes. The formulas (1) and (2) can be generalized and written as

$$S_{ij} = c\alpha_{ij} L_{ij} \left(p_{ij}\right)^b , \tag{3}$$

where S_{ij} is the leakage outflow from the pipe connecting nodes i and j, c is the leakage coefficient of proportionality for the whole network, or for pressure zone, α_{ij} is the coefficient, which is a function of pipe diameter, age and its material, and it takes into account the areas where the pipe lies. L_{ij} is the pipe length, p_{ij} is the average pressure along the pipe, and b is an exponent of pressure. The average pressure along the pipe can be approximated as

$$p_{ij} = 0.5\left(H_i - z_i + H_j - z_j\right), \tag{4}$$

or

$$p_{ij} = 0.5\left(p_i + p_j\right), \tag{5}$$

where H_i, H_j; z_i, z_j and p_i, p_j are heads, ground elevations and pressure heads in nodes i and j, respectively. Using eqn (5), the eqn (3) can be written as

$$S_{ij} = (0.5)^b c\alpha_{ij} L_{ij} \left(p_i + p_j\right)^b . \tag{6}$$

If we distribute the leakage of pipe between the terminal nodes we can write

$$S_{ij} = Q_{ij} + Q_{ji} , \tag{7}$$

where Q_{ij} is leakage from node i caused by the pipe which connects this node with node j, and Q_{ji} is leakage from node j, caused by the pipe which connects this node with node i. Let us assume that the leakage from the pipe is distributed between the terminal nodes proportionally to the pressure in the nodes. Then we have

$$Q_{ij} = (0.5)^b c\alpha_{ij} L_{ij} p_i \left(p_i + p_j\right)^{b-1} , \tag{8}$$

$$Q_{ji} = (0.5)^b c\alpha_{ij} L_{ij} p_j \left(p_i + p_j\right)^{b-1} . \tag{9}$$

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The total volume of leakage from node i is equal to

$$Q_i = cQ_i^*, \tag{10}$$

where

$$Q_i^* = \sum_{j=1}^{N_i} (0.5)^b \alpha_{ij} L_{ij} p_i (p_i + p_j)^{b-1}. \tag{11}$$

Here N_i is the total number of pipes linked to node i. The coefficients α_{ij} can be expressed in the form of

$$\alpha_{ij} = \beta_{ij} \gamma_{ij} (1 + \delta_{ij} \tau_{ij}) D_{ij}, \tag{12}$$

where β_{ij} is the coefficient of the location of the pipe, γ_{ij} is the coefficient of the material of the pipe, D_{ij} is the pipe diameter, δ_{ij} is the age coefficient and τ_{ij} is the age of the pipe. Such linear dependence of the age of the pipe is proposed by Andrés & Planells [5]. The leakage coefficient of proportionality c is determined as

$$c = \frac{k}{\sum_{i=1}^{M} Q_i^*}, \tag{13}$$

where k is the total volume of leakages and M is the total number of nodes in the network. Initially we do not know the pressure in the network with water demand and leakage. Therefore, an iteration process for the determination of pressure must be applied. For the control nodes, where the values for pressure are given, these pressures can be used during all the iteration process. For other nodes the computed pressures must be used. Thus, proceeding from the pressures in nodes which are calculated for proportionally distributed leakage, the pressures and leakages can intermittently be determined.

3 Construction of the Water Network Model for the City Centre of Tallinn

At present the number of inhabitants in Tallinn is approximately 450 000. In the past, when Estonia was a part of the Soviet Union, Tallinn developed extensively. The number of inhabitants increased and the industry grew rapidly. Since the independence of Estonia the water consumption in Tallinn has decreased over two times and is approximately equal to 100 000 m^3/day. As a result the distribution system is oversized now and it is difficult to quarantee good water quality for consumers. The water supply system of Tallinn uses

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mainly the surface water of Lake Ülemiste. The lake is connected with the surrounding waters by an open channel. Before injection to the distribution system the lake water is treated. The water network of Tallinn can be divided into several pressure zones. The main reason here is not the substantial ground level difference, but the length of pipelines. The algorithm for the distribution of leakage has been used for the network of the city centre of Tallinn. This is the most complicated part of the network, where some pipes date back to the year 1883 and some are new and unaccounted for water is equal to 28 %. In the recent years large sums have been invested into measuring consumed water in Tallinn. 89% of the houses have water meters. This serves as a good basis to evaluate the average monthly consumption with reasonable accuracy, which will be the basis for estimating the average daily consumption. For the development of the hydraulic model, the network drawings in MicroStation were used. Modelling of the network has been done on the basis of the EPANET software. The Input and Map files were constructed automatically with the developed subprograms in the MicroStation environment. For the first hydraulic calculations a simplified coarse model was applied, using pipes with diameters $\geq$ 200 mm. For making the EPS analysis the geometry of the network has been completed with smaller pipes, covering at least 90 % of network pipes. The quality of the original MicroStation drawings was inadequate. Therefore some subprograms were created to correct the uncertanties of the drawings and to enter the network data into the EPANET files. The program automatically numbers all the nodes and pipes, enters them into the INPUT file, enters all the nodes with coordinates into the MAP file and creates a database of the network pipes, which contains entries about the initial and terminal node numbers, diameters, materials and years of installation of the pipes. The used network model calibration process can be divided into two steps. In the first step we used the algorithm of distributing leakage, given in this paper. The second step will be the final model calibration by adjustment of pipe roughnesses. This subprogram is connected with the modelling package EPANET as well. Frictional headlosses had been calculated with the Hazen-Williams and Darcy-Weisbach equations. The Hazen-Williams equation has limited ranges in the hydraulic radius and pipe diameter and gives incorrect results outside its data range (Chyr Pyng Liou [6]). In our experience, especially for oversized water networks, the Darcy-Weisbach equation is more rational and applicable. The question is arising about the Colebrook-White formula, which can give incorrect values for the friction factor in the pipes with low velocities. Leakages were divided between the nodes on the basis of the algorithm (10) with (11) and (12), and this process has been realized by linking new subprograms to the EPANET software. The exponent of the pressure b = 1.18 and the pipe coefficient α_{ij} depend on the pipe diameter, age and material. The range of pipe diameters in the model is 32…1000 mm, corresponding α_1 is in the range 1.0…2.0, the age of the pipes is from 1883 to1999, the corresponding α_2 is in the range 4.0…1.0, and for steel and cast iron pipes α_3 = 1.5 and 1.0, respectively (Tallinn Water Ltd. has estimates that from all

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leakages 60...65% take place from steel pipes and 30...35% from cast iron pipes). The final pipe coefficient

$$\alpha_{ij} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \, . \tag{14}$$

This was the first estimate for the pipe coefficient which needs more calibration in the future.

4 Analysis of the modelling results

Hydraulic and water quality modelling for the city centre of Tallinn has been implemented with the software package EPANET. The network geometry is given in Figure 1. The leakage distribution algorithm improved the network model to some extent, especially at peak consumption hours. In Figure 1 network regions are given where pressure had been changed > 1 m (decreased or increased) compared with the initial calculated pressure values. The maximum node pressure increase was equal to 3.2 m and decrease 1.0 m. From the results it can be forecasted that in the network areas, where the leakages are higher, the influence of leakage distribution should be higher. In our particular case we were interested in the network area with different ages of pipes and where we had more pressure measurement results. In the area of the oldest pipes the pressure increased when the algorithm was used. The iterative process of the calculations converges well, and the convergence of the leakage coefficient proportionality c in one particular calculation is given in Figure 2. Examples of the calculations of the leakage distribution and pressure in some nodes are presented in Figures 3 and 4. The results are given for different conditions: A - leakage losses are distributed proportionally with the consumption in the nodes; B – improved distribution of leakage with the algorithm (10) with $\alpha_{ij} = 1.0$ and C – improved distribution with the influence of calculated α_{ij} (14). In Figure 4 some measured pressure values were added. The calculated results are for the average consumption hour at 6:00 a.m. The observed nodes were from two network areas, indicated in Figure 1 with **a** and **b**. The area **a** consists mainly of old pipes, several pipes had been installed over 100 years ago, among them the first pipes already in 1883. The influence of the algorithm on the older network area is more noticeable. The Standard Deviation between the measured and calculated pressures for four time moments are given in Figure 5. The average day consumption is at 6:00 and maximum at 21:00. The results A, B and C were obtained from the calculations at 6:00 by multiplying with consumption time pattern value at 10:00, 14:00 and 21:00 correspondingly. The STDEV value D at 21:00 was calculated differently, first the consumption was evaluated by using the time pattern value at 21:00 and then followed the distribution of the leakage losses with the algorithm. The influence of the time factor is significant.

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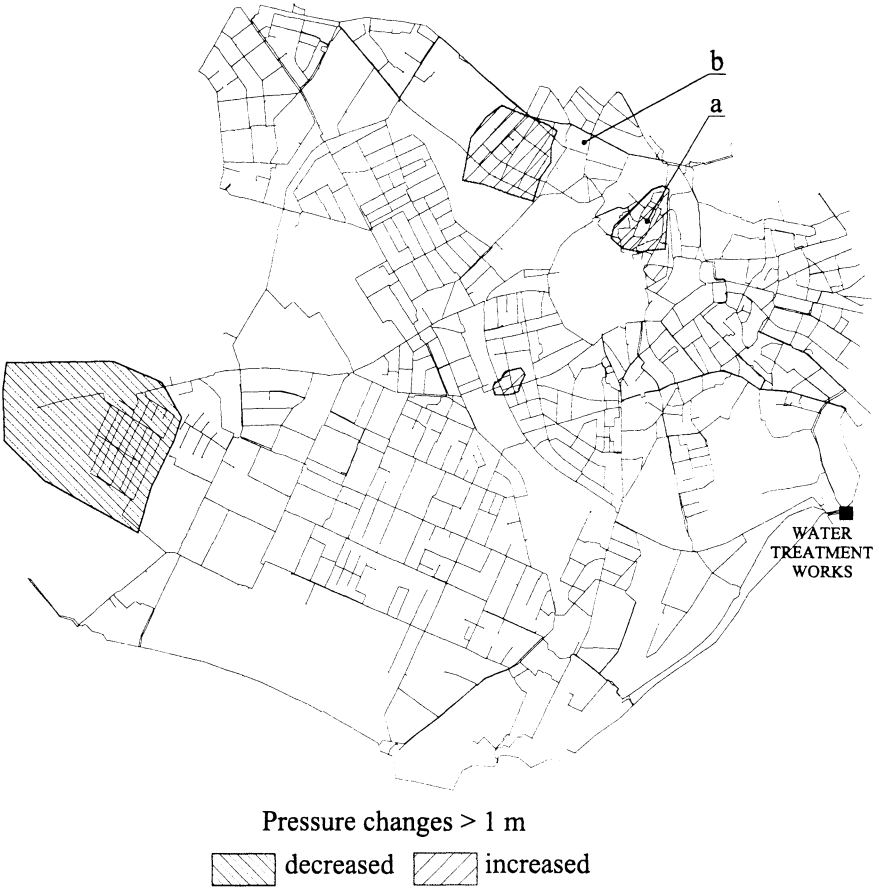

Figure 1: Network of the City Centre of Tallinn.

5 Conclusions

A model of leakage distribution is proposed, which takes into account the length, mean pressure, age, diameter and material of pipes. The iteration process is used to calculate alternately the leakage and pressure value. The EPANET software package calculates pressure distribution and the developed subprograms redistribute leakage value. The given algorithm diminishes the difference between the measured and calculated pressures, especially at the peak consumption hours. The pressure difference in the network affects the leakage distribution most.

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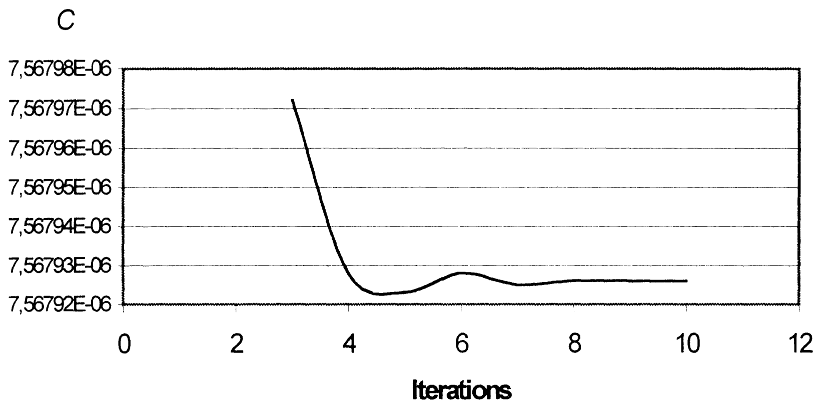

Figure 2: Convergence of the leakage coefficient of proportionality *c*.

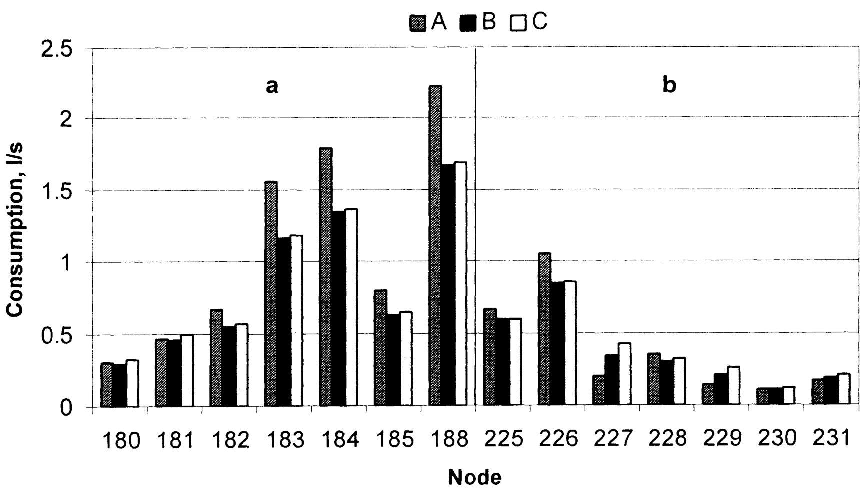

Figure 3: Water consumption in some network nodes: A – proportional distribution of leakage; B – improved distribution of leakage with α_{ij} = 1.0; C-improved distribution of leakage.

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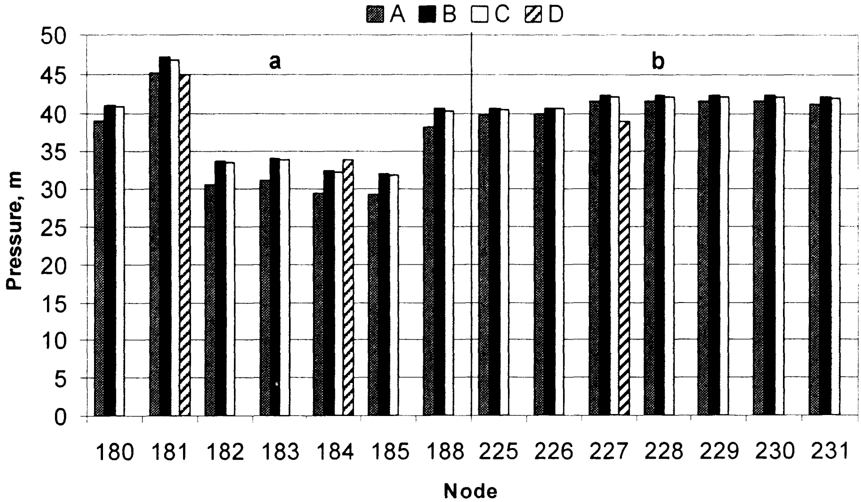

Figure 4: Calculated pressure in some network nodes: A – proportional distribution of leakage; B – improved distribution of leakage with α_{ij} = 1.0; C-improved distribution of leakage; D-measured pressure.

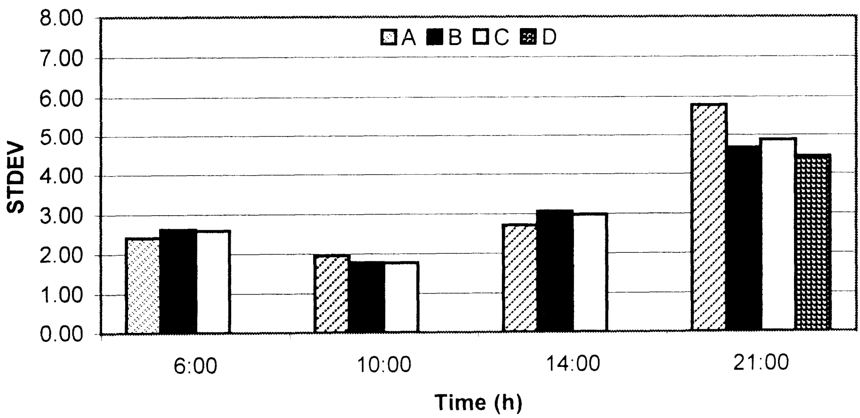

Figure 5: Standard deviation of calculated pressure in 48 pressure measurement nodes: A – proportional distribution of leakage; B - improved distribution of leakage with α_{ij} = 1.0; C - improved distribution of leakage; D- time pattern at 21:00 is applied.

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