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An Optimization Strategy for Water Distribution Networks

Önder Ekinci · Haluk Konak

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Abstract An optimization strategy based on head losses minimization is developed for the least cost design of water distribution networks. A new weighting approach is suggested for calculating the initial flow distribution and optimum pipe diameters of the weighted flow distribution is presented by using least square method. In the mean time homogenous and isotropous head losses are maintained with implications of head loss path choice. The model is employed for designing and/or modifying pipe sizes while the classical Hardy-Cross network solver is used to balance the flows. The whole algorithm is programmed and applied to a two-looped network selected from the literature and the results are presented on a comparative basis. A FORTRAN software with the necessary steps in the flow chart is written for the optimization calculations in this paper.

Keywords Algorithms · Optimization · Water distribution · Network · Cost

1 Introduction

There are many models that have been developed to solve water distribution network optimization non-linear problem using different techniques. Among them are the deterministic, dynamic, linear, non-linear, enumeration etc. approaches, in addition to heuristic techniques such as genetic algorithm (Savic and Walters 1997), simulated annealing (Cunha and Sousa 1999, 2001), harmonic search (Geem et al. 2000, 2002, Geem and Tseng 2002), ant colony optimization algorithm (Maier et al. 2003) and shuffled frog leaping algorithm (Eusuff and Lansey 2003).

The objective functions focus usually on the cost minimization while the reliability or precision of the optimal solutions are not assessed sufficiently. Reliability is the ability of a system or component to perform its required functions under stated conditions for a specified period of time whereas the precision is the state or quality of being exact.

Ö. Ekinci (✉)
Asım Kocabıyık Vocational School, University of Kocaeli, Hereke, 41800 Kocaeli, Turkey
e-mail: oekinci@kou.edu.tr

H. Konak
Faculty of Engineering, Deparment of Geodesy and Photogrammetry Engineeering,
Information Technologies Research Center, University of Kocaeli, 41100 Kocaeli, Turkey
e-mail: hkonak@kou.edu.tr

Recently, several papers addressed the issue of reliability (or redundancy) of a water distribution networks, together with cost minimization and adopting a multi-objective approach to solve the problem (Tolson et al. 2004, Prasad and Park 2004, Farmani et al. 2005, Dandy and Engelhardt 2006).

Among the main difficulties in use of such models are insufficient solutions for the application problems (Walski 2001) and inability to provide better alternatives as compared to the traditional methods (Goulter 1992). The main precision problem is how to evaluate the initial parameters, flows and diameters. If decision makers use different initial value sets, they can find occasionally not global but local optimum. The solution of non-linear equations could be performed by consecutive approaches in such a manner that the results in the first stage should not be far from the target values. Hence, estimation of the initial parameters is one of the main problems, although it does not seem to be an important process at the beginning.

In any water distribution network system design, one of the major objectives is to minimize the head losses along the pipelines. The most important constraint is to get a solution with the minimum cost. Therefore, a minimum head loss optimization strategy, which could provide a rapid, realistic and practicable solution, is developed based on a precision process. Such a model provides a reliable solution for the investigation of the optimal flow distributions and corresponding diameters.

In this paper a weighted approach is introduced to determine the initial pipe flows as the most suitable and the most approximate calculation for the real case. Flows and diameters are estimated initially while the heads are taken as variables. The proposed approach is applied successfully to the Alperovits and Shamir (1977) two-looped network system and data. The weighted flow rate pre-balance by Hardy–Cross solver is imposed among the initial values. Optimum diameter determination according to the initial flow rates is performed by a weighted least square method.

2 Model Formulation

2.1 The Determination of the Objective Functions for the Weighting Optimization

The Darcy–Weisbach functional relation gives the head loss, h_i, at i-th pipe as,

$$h_i = \frac{8}{\pi^2 g} f_i \frac{L_i}{D_i^5} Q_i^2 \tag{1}$$

where g is the acceleration of gravity; f_i is the friction factor of pipe, which can be calculated using the Colebrook–White formula L_i is the pipe length, D_i is the pipe diameter and Q_i is the discharge in pipe i.

2.1.1 Homogeneously Distributed Head Losses

Equation 1 is difficult to use in the case of pipe networks, and therefore, it is convenient to write it similar to the Chezy–Manning formula as,

$$h_i = K_i Q_i^2 \tag{2}$$

where K_i is a constant. If the corrections are aimed to be in the normal distribution, the functional model would be arranged by its unknowns as,

$$\frac{1}{\sqrt{K_i}} \sqrt{h_i} = Q_i \tag{3}$$

Perturbation of variables h_i and Q_i in Eq. 3 after the ignorance of non-linear terms leads to the following expression where h_i is taken as the average values of the unknowns.

$$\frac{1}{\sqrt{K_i}}\sqrt{h_i} + \frac{1}{2\sqrt{K_i h_i}}dh_i = Q_i + v_i \tag{4}$$

Herein, v_i indicates the correction amount (error due to perturbation) in the discharge. Multiplication of both sides in Eq. 2 by h_i yields $K_i h_i = K_i^2 Q_i^2$, acceptance of $h_{i_0} \cong 0$ and their substitution into Eq. 4 leads to,

$$\frac{1}{2K_i Q_i}dh_i = Q_i + v_i \tag{5}$$

with the correction expression as,

$$v_i = \frac{1}{2K_i Q_i}dh_i - Q_i \tag{6}$$

In order to determine the most suitable estimation of the unknowns and the most probable corrections of the measurements, it is necessary to have the sum of the squares of the corrections as minimum,

$$\underline{v}^T \underline{v} \Rightarrow \min \tag{7}$$

where

$$\underline{v}^T = [v_1, v_2, \ldots, v_n]$$

is the correction vector. Equation 7 yields a square matrix with diagonal elements

$$\underline{\mathrm{A}} = \left[\mathrm{a}_{1,1}, \mathrm{a}_{2,2}, \ldots, \mathrm{a}_{\mathrm{n,n}}\right]$$

and coefficients as

$$a_{i,i} = \frac{1}{2K_i Q_i}$$

The collection of head losses in pipes appears in the form of an unknown vector as,

$$\underline{x}^T = [dh_1, dh_2, \ldots, dh_n]$$

Additionally by considering the flow measurement vector as

$$\underline{\ell}^T = [Q_1, Q_2 \ldots, Q_n]$$

the normal equations can be established through the error propagation law as,

$$\underline{A}^T \underline{Ax} - \underline{A}^T \underline{\ell} = 0 \tag{8}$$

or

$$\underline{x} = \left(\underline{A}^T \underline{A}\right)^{-1} \underline{A}^T \underline{\ell} \tag{9}$$

Constraint functions are established among the unknowns (head losses) for calculating the flow corrections (v_i). As a restriction in each loop as a condition the total head loss should be equal to zero. From the aforementioned concepts such a restriction requires that

$$h_1 - h_2 = K_1(Q_1 + \Delta Q)^n - K_2(Q_2 - \Delta Q)^n = 0 \tag{10}$$

Equations 8 and 10 indicate that the head loss values are directly dependent on the unknown coefficients. If the flow distribution and the corresponding pipe diameters are chosen appropriately, then the head losses will be homogeneous accordingly. In an ideal situation the head losses are expected to be almost equal.

Eigenvalues, λ_i, are the most important criterion of precision in a loop optimization. The coefficient matrix, $\underline{A}^T\underline{A}$, is a diagonal matrix, and therefore its eigenvalues are equal to its diagonal elements, which mean that

$$\lambda_i = \frac{1}{4K_i^2 Q_i^2} \tag{11}$$

This expression can be rewritten by considering Eq. 2 as

$$\lambda_i = C_i / K_i Q_i^2 = C_i / h_{k_i} \tag{12}$$

where $C_i = 1/4K_i$. According to this relation, there is only a simple linear conversion between the eigenvalue and the head loss. A favorable objective function can be chosen by considering the precision requirements as one of the following alternatives.

1) The maximum head loss should be minimum,

$$\max\{h_i\} \Rightarrow \min \tag{13}$$

2) The head losses should have the same values (isotropous),

$$\max\{h_i\} = \min\{h_i\} \tag{14}$$

3) The distribution of head losses should be symmetrical (homogeneous):

$$\frac{\max\{h_i\}}{\min\{h_i\}} \leq \varepsilon \tag{15}$$

where ε is an acceptable ration value. The head losses are dependent on the pipe diameters rather than the initial flow distribution, which can be expressed by considering Eqs. 1 and 2 as,

$$K_i = f_i L_i c / D_i^5 \tag{16}$$

where $c = 8/\pi^2 g$ is a constant. This last expression shows that if the pipe diameter increases, the head loss decreases. Accordingly, the cost increases, if a cost function is constituted with the commercial pipe diameters, then the cost increases also with the quality.

2.1.2 An Isotropous Head Loss Distribution for Whole Network

In many optimization procedures, the precision criteria with global objectives representing whole network are preferred instead of those with scalar objectives. A homogeneous and isotropous matrix can be chosen as the objective function for an ideal water distribution network. For this purpose, first an isotropous criterion matrix is established as the objective function as,

$$\underline{Q}_x = c_e \underline{E} \tag{17}$$

where c_e is a constant and E is a unit matrix, and the functional model can be established as,

$$\left(\underline{A}^T\underline{A}\right)^{-1} \doteq \underline{Q}_x \tag{18}$$

where $\doteq$ is the inconsistency sign. If the problem is converted to an inverse of criterion matrix, then the basic model could be obtained with direct approach (Konak 1994, Grafarend et al. 1979).

$$\underline{A}^T\underline{A} \doteq \underline{Q}_x^{-1} \tag{19}$$

which can be written explicitly as,

$$\begin{bmatrix} \frac{1}{4K_1^2Q_1^2} & 0 & 0 & \cdots & 0 \\ & \frac{1}{4K_2^2Q_2^2} & 0 & \cdots & 0 \\ & & \frac{1}{4K_3^2Q_3^2} & \cdots & 0 \\ & & & \ddots & \frac{1}{4K_n^2Q_n^2} \end{bmatrix} \doteq \begin{bmatrix} 1 & 0 & 0 & \cdots & 0 \\ 0 & 1 & 0 & \cdots & 0 \\ 0 & 0 & 1 & \cdots & 0 \\ \vdots & & & & \\ & & & & \\ 0 & 0 & 0 & \cdots & 1 \end{bmatrix} \tag{20}$$

This expression indicates clearly that the unknowns are K_i coefficients. By definition as,

$$p_i = 1/K_i^2 \tag{21}$$

for weights on a_i which are the diagonal elements of coefficients matrix one can write that

$$\underline{B}^T\underline{PB} \doteq \underline{E} \tag{22}$$

with

$$\text{diagonal}\,(\underline{B}_i) = \left[\frac{1}{2Q_1}, \frac{1}{2Q_2}, \ldots, \frac{1}{2Qn}\right] \tag{23}$$

On the other hand, provided that

$$\left(\underline{B}^T\Theta\underline{B}^T\right)\underline{p} \doteq \underline{q} \tag{24}$$

then the weights can be calculated as,

$$\underline{p} \doteq \left(\underline{B}^T\Theta\underline{B}^T\right)^{-1}\underline{q} \tag{25}$$

where $\underline{p}$ is the vector ($\underline{P}$), $\underline{q}$ is the vector ($\underline{E}$), Θ is the Khatri–Rao multiplication.

Equation 22 can be solved by the least square method (LSM). If (*) is Hadamard multiplication, then the LSM solution will be as,

$$\left(\underline{BB}^T*\underline{BB}^T\right)\underline{p} - \left(\underline{B}^T\Theta\underline{B}^T\right)\underline{q} = 0 \tag{26}$$

The residue matrix is defined as,

$$\underline{D} = \left(\underline{B}^T\underline{B}\right)^{-1} - \underline{Q} \tag{27}$$

with residue vector as while d=vector (D)

$$\underline{d}^T\underline{d} \Rightarrow \min \tag{28}$$

By dealing with the basic Eq. 22, it is easily seen that the criterion matrix is the solution of a simple normal equation system with $(n \times n)$ dimensions when it is a diagonal matrix. If the system is solved for each pipe, than the following equations can be obtained as,

$$\frac{1}{4Q_i^2} p_i = q_i \tag{29}$$

and

$$p_i = 4Q_i^2 q_i \tag{30}$$

From Eq. 21, the p_i and K_i coefficients are obtained as,

$$p_i = \frac{1}{K_i^2} \tag{31}$$

and

$$K_i = \frac{1}{\sqrt{p_i}} \tag{32}$$

respectively, K_i are the most suitable coefficients for the isotropous structured head loss distribution. Hence, if the commercial diameters are to be found, then weight coefficients for the maximum pipe diameter D_0 should be calculated for each pipe by Eqs. 33 and 34 as,

$$K_0 = f_0 L_0 \frac{c}{D_0^5} \tag{33}$$

and

$$K_i = f_i L_i \frac{c}{D_i^5} \tag{34}$$

where f_0 and f_i are friction factors. The ratio of these two last expressions leads to,

$$\frac{K_i}{K_0} = \frac{f_i L_i c D_0^5}{f_0 L_0 c D_i^5} \tag{35}$$

where $\frac{f_i}{f_0}$ is a favorable coefficient and it can be taken as almost equal to 1 $\left(\frac{f_i}{f_0} \cong 1\right)$. Hence, the optimum diameters could be obtained from Eq. 36 as,

$$D_i^5 = \frac{K_0 f_i L_i}{K_i f_0 L_0} D_0^5 \tag{36}$$

If the head loss distribution is calculated by balancing the head losses with the determined commercial pipe diameters then a homogeneous (symmetrical) head loss distribution can be obtained insofar. In addition, the optimum pipe diameters for the initial flow distribution could be determined, while the head loss distributions are controlled in the optimization procedure.

3 A Minimum Head Loss Optimization Strategy

The minimum head loss optimization strategy includes three steps as the optimization of precision, reliability and cost. Herein, the cost and reliability optimizations are especially important. However, the optimization of precision seems to be necessary for all the stages. Therefore, cost and reliability optimization processes should be performed simultaneously. In this paper, a FORTRAN software with flow chart in Fig. 1 is developed for the optimization strategy.

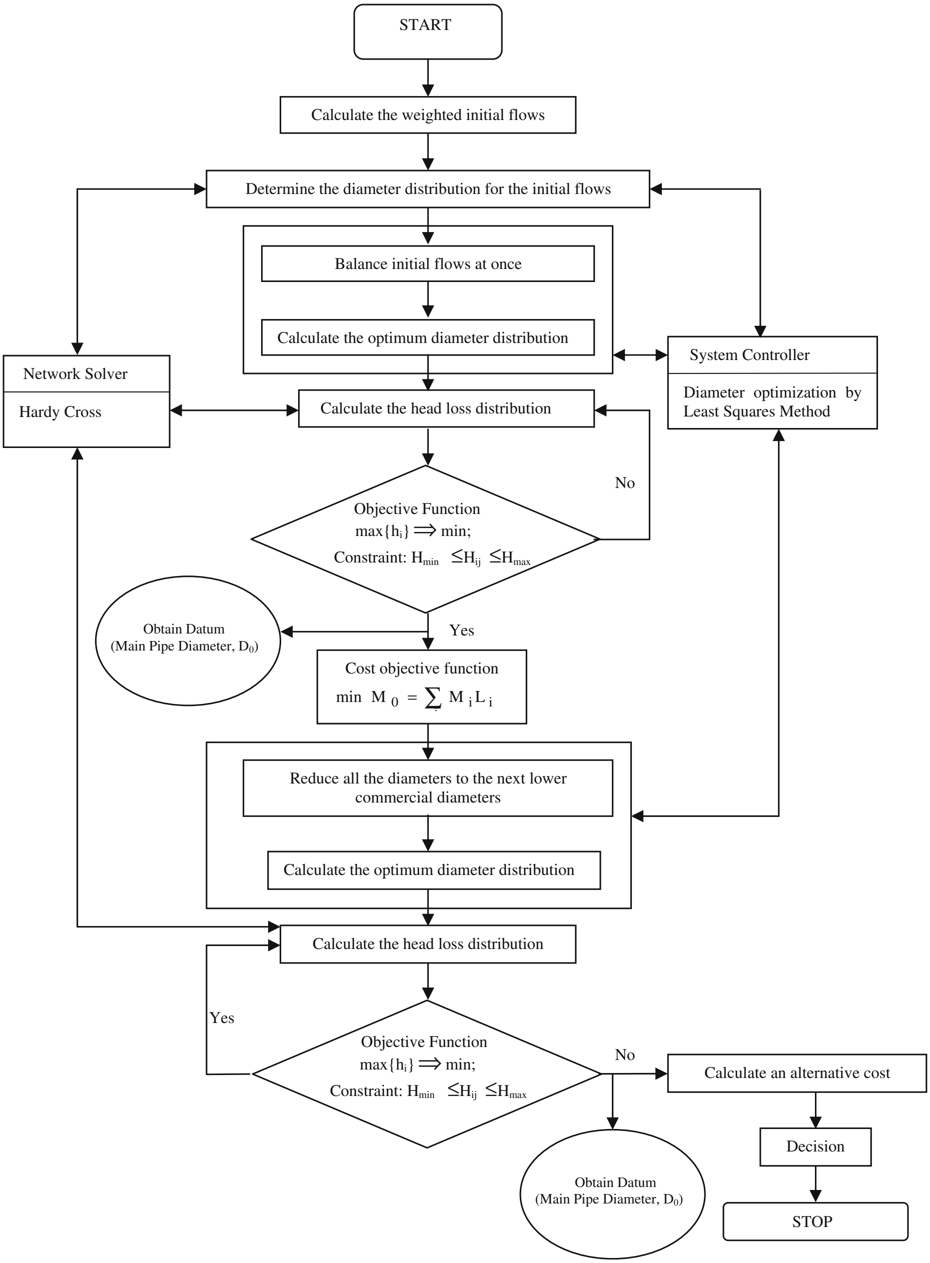

Fig. 1 The flow chart for the minimum head-loss optimization strategy

The optimization of precision procedure is applied to the network by choosing the minimization of the maximum head loss as the priority objective function as

$$\max\{h_{f_i}\} \Rightarrow \min$$

then the network is taken to its optimum condition with respect to the precision. Under these conditions, the objective function is performed as the minimization of the total pipe cost

$$\sum M_i(D, L) \Rightarrow \min$$

Here, the most important constraint is the pressure constraint which is taken as $30 \leq H_i \leq$ 80 m.

4 Weighted Flow Distribution

If a decision-maker chooses initial flow distribution based on his/her expert view then it is highly probable that the solution algorithm converges to a local optimum. In this case, the number of iterations increases and with the change of the initial flow distribution, another local optimum can be obtained. The significance of initial flow distribution determination, and its effects on the solution can be seen in the numerical applications clearly (Goulter et al. 1986). Morgan and Goulter (1985) proposed the first weighting algorithm for flow distribution. Their algorithm balanced by Hardy–Cross method, the ratio between the flow transferred by a pipe to the node and the total flow at the node is accepted as the weight of the pipe related to the node. In other studies, the initial flow distribution is chosen according to the practice of decision maker, but the criteria for the selection of initial flow distribution are not explained clearly.

According to the weighting algorithm based on a loop approach in this study, the sum of the flows drawn from the beginning and the end of the link (pipelines) is rated to the sum of the flows drawn from the total network then the weighted flow is obtained. If this pre-weight is taken independently, a deceptive situation that is not in accordance with the reality may appear such that the weight of the pipes following the main pipe may be less than that of the pipes at the end of the network. In this paper, the sum of the pipe weight and the weights of the consecutive pipes fed by this pipe in the network expresses the real weight of this pipe in the network. The main steps for this algorithm are as follows

1. The path choice number, s, is constituted
2. The weights of each pipe are calculated for the path choice. If q_i is the discharge flow at the node i, and q_{ij}^s is the average discharge flow at the pipe, which connects i, j nodes then,

$$q_{ij}^s = (q_i + q_j)/2 \tag{37}$$

and the weighted flow of the connection pipe from i to j nodes $\left(a_{ij}^s\right)$ can be calculated as,

$$a_{ij}^s = \frac{q_{ij}^s}{\sum q_{ij}^s} \tag{38}$$

with the control condition that

$$\sum a_{ij}^{s} = 1 \tag{39}$$

3. The weighted flows, Q_{ij}, of the successive pipes for the path choice can be calculated as,

$$Q_{ij}^{r,k+1} = \left(Q_{ij}^{r,k} - q_i\right) \sum a_{ij}^{r} \tag{40}$$

where

$$Q_{ij} = Q_{ij}^{r,k+1} \tag{41}$$

Herein, r is the number of successive pipes at the path, and k is the number of the calculation steps.
4. The steps 2 and 3 are repeated for each path choice.

5 The Determination of the Optimum Path

The determination of the consecutive pipes fed by a pipe and the calculation of the heads at the nodes require the determination of optimum path in the network. After the determination of weighted flow distribution, the flows are obtained in a consistent manner related to the network conditions. The Hardy–Cross network solver balances this pre-initial flow distribution to get the optimum initial flow distribution. In this manner, the first step for optimum solution with a few iterations can be obtained by initial flow distribution determination. This approach forms a basis for the choice of optimum path, which is then determined by the application of the minimum head loss optimization strategy including head loss distribution, distribution of diameters for the main pipe and the other pipes and the trials for the determination of the path with the minimum cost.

5.1 Optimization of Diameters

Since the pipe diameters are one of the input parameters in a solution strategy, the initial once-balanced flows are not sufficient alone for the solution. The optimum initial diameter determination according to the initial flows is a significant problem. It is essential that a solution should be found for the pipe diameters' determination that are consistent with the network conditions, according to the distribution of optimum initial once-balanced flows.

The proposed methodology in this paper includes a weighting optimization process by the LSM with a criterion matrix in a completely isotropic structure, which represents the whole network and it is used as the objective function. Since the initial value (input parameter) is the diameter of the main pipe (datum) in the diameter optimization process, the optimum diameter for the system could be obtained after few iterations starting from the maximum commercial diameter possible in the system. Thus, an initial diameter, D_0, of main pipe could be obtained as the most approximate value.

By the application of diameter optimization for searching the initial, optimized and alternative solutions, both the optimum diameters could be determined, and the control of

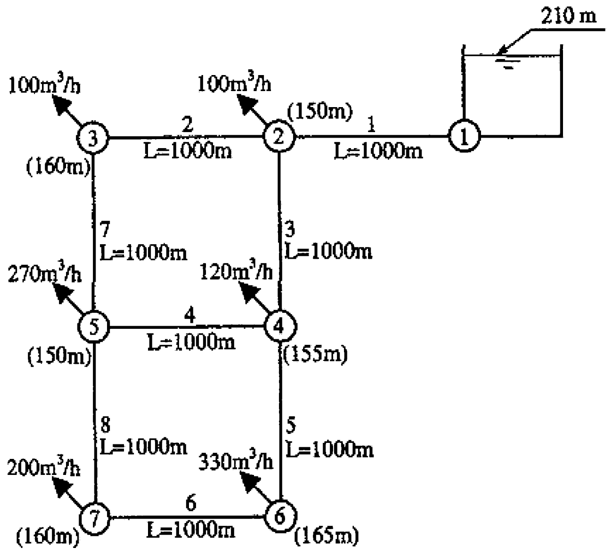

Fig. 2 The two-looped network of Alperovits and Shamir (1977)

the distribution of head losses become possible in the system. In this manner, the decision-maker could obtain the optimum solution with the least cost rapidly by reducing the number of iterations, for example, from 122 to 6.

5.2 Balancing the Distribution of Head Losses

If the head loss is aimed to be minimum in each pipe, increasing the diameter of the pipe at the maximum head loss location with the following upper commercial diameter in mind until the pressure constraint is satisfied, should reduce the head loss and consequently the distribution of head losses should be homogenized. In this manner, the pressure constraint is satisfied by changing the pipe diameters in the algorithm. Provided that that the pressure is not satisfied at any node in the network, the pipe diameter under the maximum head loss is increased to the next upper diameter, and this process continues until the pressure constraint is satisfied at each node. The head losses distribution is homogenized for approximating the ideal solution. It should be noted that the cost function tends downward when the optimization of diameters is performed.

5.3 Searching the Alternative Solutions

Since there are many local optimums in the solution space of non-linear problems, the probability of the presence of more optimal solutions should be concerned also. Although the decision-maker could make some changes interactively for searching such solutions, it is difficult to reach an optimum solution converging to the real value by this way, when the errors caused by the increase in the number of iterations and choices of the decision-maker are taken into account. Therefore, the changes should not be local and they should include all elements in the system. In the approach proposed here, search for reducing all the diameters performs a better optimal solution determination as optimum in the first run to the next lower commercial diameters. Hence, it could be said that the system solution is leaped from the local optimum. It is observed in the numerical applications, in general, that the alternative solution does not differ from the optimum solution.

6 Numerical Applications

For the comparison of the proposed model with widely accepted ones in the literature, it is necessary to consider objectivity, consistency, sufficiency and reliability in the final testing. In this study, the proposed model is compared with the Alperovits and Shamir (1977) two-looped network which provides enough data for testing (see Fig. 2).

In a water distribution network, the aim is to get the head losses through the pipelines connecting the pairs of the nodes as minimum and equal as possible. In addition, the cost of design is required to be minimum with respect to the commercial pipe diameters. On the other hand, the design solution has to satisfy the constraints of pressure, velocity, etc. For a minimum cost optimization solution procedure should follow a strategy including the following four steps depending on and controlling each other.

1. Optimization of the initial flows (weighted flow distribution),
2. Determination of the initial pipe diameters: diameter optimization (datum optimization),
3. Determination of the optimum path choice according to the first two steps, and
4. The head loss optimization to control the successive steps above.

6.1 Testing of the Model and Reliability

Alperovits and Shamir (1977) presented a stable network model with equal pipe lengths that are really connected at right angles and hence it is possible to observe the head loss distribution through the network in a reliable manner.

The diameter optimization procedure on the initial flows that are obtained by the initial data group is formed by the weighted flow distribution approach. The flow distribution from the commercial diameter optimization procedure may not be effective to minimize the inconsistency effect.

In the numerical application, the proposed model is applied to the path choices determination according to nodes of 5 (the first path) and 7 (the second path) and the datum are observed as 20 and 18 in. respectively. The results are clearly compatible with the results of Alperovits and Shamir (1977) as shown in Table 1.

For models given in the literature, it is possible to see the pipe diameters as low as 1 in., which is not used in practical use. The proposed strategy produces the minimum pipe diameter as 4 in. which can be used in practice and it provides a homogenous head loss distribution with plausible commercial pipe diameters in addition to solutions with less cost as in Tables 2 and 3.

6.2 The Comparisons of the Results

The optimum design cost solutions of the two-looped test network in the literature gives the unit costs as 479.525 (Alperovits and Shamir 1977), 435.015 (Goulter et al. 1986), 417.500 (Kessler and Shamir 1989), 415.271 (Fujiwara and Khang 1990), 402.352 (Eiger et al.

Table 1 Datum and diameter optimization results

	Node 5												Node 7											
	Initial flows of Alperovits and Shamir (1977)						Weighted initial flows						Initial flows of Alperovits and Shamir (1977)						Weighted initial flows					
	Optimum solution			Alternative solution			Optimum solution			Alternative solution			Optimum solution			Alternative solution			Optimum solution			Alternative solution		
No	D_0	D_1	D_i	D_0	D_1	D_i	D_0	D_1	D_i	D_0	D_1	D_i	D_0	D_1	D_i	D_0	D_1	D_i	D_0	D_1	D_i	D_0	D_1	D_i
1–2	355.6	355.6	508.0	457.2	355.6	508.0	406.4	406.4	508.0	457.2	406.4	457.2	406.4	406.4	457.2	406.4	406.4	457.2	406.4	406.4	457.2	406.4	406.4	457.2
2–3	254.0	254.0	304.8	254.0	254.0	304.8	304.8	304.8	304.8	254.0	304.8	304.8	304.8	304.8	304.8	254.0	304.8	304.8	355.6	355.6	355.6	304.8	355.6	355.6
2–4	355.6	304.8	406.4	355.6	304.8	406.4	355.6	355.6	406.4	355.6	355.6	355.6	355.6	355.6	406.4	355.6	355.6	406.4	355.6	355.6	355.6	304.8	355.6	355.6
4–5	152.4	203.2	203.2	152.4	203.2	203.2	254.0	254.0	254.0	203.2	254.0	254.0	203.2	152.4	152.4	76.2	152.4	152.4	254.0	203.2	203.2	152.4	203.2	203.2
4–6	304.8	304.8	355.6	304.8	304.8	355.6	355.6	355.6	355.6	304.8	355.6	355.6	355.6	355.6	355.6	304.8	355.6	355.6	304.8	304.8	304.8	254.0	304.8	304.8
6–7	254.0	254.0	254.0	203.2	254.0	254.0	304.8	254.0	254.0	203.2	254.0	254.0	304.8	304.8	304.8	254.0	304.8	304.8	152.4	203.2	203.2	152.4	152.4	152.4
3–5	203.2	254.0	254.0	203.2	254.0	254.0	254.0	304.8	304.8	254.0	304.8	304.8	254.0	304.8	304.8	254.0	304.8	304.8	355.6	304.8	304.8	254.0	304.8	304.8
5–7	203.2	152.4	152.4	152.4	152.4	152.4	152.4	101.6	203.2	152.4	254.0	254.0	254.0	304.8	304.8	254.0	304.8	304.8	254.0	254.0	254.0	203.2	254.0	254.0
Cost	296	295	473	345	295	473	390	397	507	366	406	476	397	421	491	336	426	496	400	388	428	309	376	416
Approximate datum	355.6						406.4						406.4						406.4					
Optimum datum	508.0						457.2						457.2						457.2					

Table 2 The trend of cost function

Approximate datum		508.0		457.2		406.4		355.6		304.8		254.0	
Optimum datum		508.0		508.0		457.2		457.2		508.0		508.0	
Path	Initial flows	Optimum solution	Alternative solution	Optimum solution	Alternative solution	Optimum solution	Alternative solution	Optimum solution	Alternative solution	Optimum solution	Alternative solution	Optimum solution	Alternative solution
Node 5	1	682	690	544	514	476	476	473	473	480	480	491	491
	2	722	690	544	514	507	476	482	482	480	480	491	491
Node 7	1	723	713	553	572	491	496	507	507	500	500	529	529
	2	676	675	535	505	428	416	435	435	435	435	440	440

1 Initial flows of Alperovits and Shamir (1977), *2* weighted initial flows

Table 3 Minimum head loss optimization strategy

	Optimization of head loss distributions													Optimization of head loss distributions for alternative solution											
	1	2				3				4				5				6				7			
No	Q_0 m^3/h	D_0 mm	Q_1 m^3/h	h_i m	H_i m	D_i mm	Q_i m^3/h	h_i m	H_i m	D_i mm	Q_i m^3/h	h_i m	H_i m	D_t mm	Q_i m^3/h	h_i m	H_i m	D_i mm	Q_i m^3/h	h_i m	H_i m	D_i mm	Q_i m^3/h	h_i m	H_i m
1–2	−1120	406.4	−1120	11.3	48.7	406.4	−1120	11.3	48.7	457.2	−1120	6.09	53.9	406.4	−1120	11.3	48.7	406.4	−1120	11.3	48.7	457.2	−1120	6.09	53.9
2–3	−554.6	355.6	−457.7	3.78	34.9	355.6	−436.3	3.43	35.3	355.6	−436.3	3.43	40.5	304.8	−436.3	7.67	31.1	355.6	−432.2	3.36	35.4	355.6	−432.2	3.37	40.5
2–4	465.9	355.6	561.7	5.69	38.1	355.6	584.1	6.14	37.6	355.6	584.1	6.14	42.8	304.8	584.1	13.7	30.0	355.6	587.2	6.21	37.5	355.6	587.2	6.22	42.7
4–5	−76.5	254.0	61.2	0.39	38.9	203.2	74.4	1.84	36.8	203.2	74.4	1.82	41.9	152.4	61.2	5.59	18.3	203.2	69.3	1.60	36.7	203.2	69.3	1.60	41.9
4–6	345.6	304.8	304.8	3.74	29.3	304.8	313.9	3.97	28.6	304.8	313.9	3.97	33.8	254.0	327.2	11.2	13.9	304.8	322.1	4.18	28.3	304.8	322.1	4.19	33.5
6–7	15.3	152.4	−25.5	0.98	26.7	203.2	−16.3	0.09	25.7	203.2	−16.3	0.08	30.9	152.4	−3.1	0.02	14.2	101.6	−8.2	0.75	25.2	101.6	−8.2	0.75	30.3
3–5	−454.7	254.0	−357.8	2.31	42.7	304.8	−336.4	4.55	40.8	304.8	−336.4	4.45	45.9	254.0	−336.4	11.8	29.3	304.8	−332.3	4.45	40.9	304.8	−332.3	4.45	46.1
5–7	−110.1	254.0	−149.9	2.36	26.9	254.0	−140.7	2.08	26.6	254.0	−140.7	2.07	31.7	203.2	−128.4	5.50	8.4	254.0	−132.5	1.83	26.5	254.0	−132.5	1.83	31.7
Cost		400				388				428				309				376				416			
Iteration number		3				5				1				5				4				1			
Precision criterions																									
h_{max} (m)		11.25				11.25				6.14				13.72				11.25				6.22			
h_{min} (m)		0.392				0.087				0.85				0.015				0.75				0.75			
h_{max}/h_{min}		28.71				128.94				7.256				928.6				15.04				8.28			
		$\sum h_i = 30.50$m				$\sum h_i = 33.34$				$\sum h_i = 28.16$				$\sum h_i = 66.73$				$\sum h_i = 33.66$				$\sum h_i = 28.50$			

Table 4 Solutions for the Alperovits and Shamir (two-loop) network

Literature	Solutions	Cost (units)	Pipe diameter (mm)	
			Min	Max
Alperovits and Shamir 1977	Linear programming	497.525	101.6	508.0
Goulter et al. 1986	Linear programming	435.015	25.4	508.0
Kessler and Shamir 1989	Linear programming	417.500	50.8	457.2
Fujiawara and Khang 1990	Linear programming	415.271	25.4	457.2
Eiger et al. 1994	Linear programming	402.352	25.4	457.2
Loganathan et al. 1995	Simulated annealing	412.931	25.4	457.2
Savic and Walters 1997	Genetic algorithm	419.000	25.4	457.2
Sherali and Smith 1997	Linear programming	436.684	76.2	457.2
Sherali et al. 1998	Linear programming	436.915	25.4	457.2
Abebe and Solomatine 1998	Genetic algorithm	419.000	25.4	457.2
Cunha and Sousa 1999	Simulated annealing	419.000	25.4	457.2
Todini 2000	Resilience index	419.000	25.4	457.2
Geem et al. 2002	Harmonic search	419.000	25.4	457.2
Eusuff and Lansey 2003	Shuffled frog leaping algorithm	419.000	25.4	457.2
Prasad and Park 2004	Genetic algorithm	419.000	25.4	457.2
Present model	Minimum head loss strategy	416.000	101.6	457.2

1994), 405.381 (Loganathan et al. 1995), 436.684 (Sherali and Smith 1997), 436.915 (Sherali et al. 1998), and 419.000 (Savic and Walters 1997, Abebe and Solomatine 1998, Cunha and Sousa 1999, Todini 2000, Geem et al. 2002, Eusuff and Lansey 2003, Prasad and Park 2004). On the other hand, the proposed model produces the cost value of 416.000 units with less number of iteration (Ekinci 2003, Ekinci and Konak 2005; Table 4).

The minimum head loss strategy is not aimed only at the minimum costs due to its objective functions, which means that the main object of the optimization is to provide expectations of its objective functions sufficiently while the design cost reaches to an optimum value. The design cost is one of the most less cost value than the others. Another important result from the proposed model is the minimum pipe diameter value as 101.6 mm (4 in.) which is obtained from the model solution that can be used in practice. This situation shows that the model yields also a homogenous pipe diameter distribution as in Table 1 (Ekinci 2003, Ekinci and Konak 2005). Among others only Alperovits and Shamir (1977) study gave 4 in. as the minimum pipe diameter where design cost is the largest in the literature.

If the problem is taken from the objective functions and constraints points of view then the precision degree of the proposed model is acceptable within the expectations of decision makers (Tables 2 and 3). This situation shows the model's interior consistency.

7 Conclusions

In design of the water distribution network, the head losses through the pipelines should be minimum and nearly equal as possible as they can be. In this study, the initial flows and pipe diameters are determined by a weighting optimization process to get a reliable solution satisfying these conditions in a concrete manner. A datum optimization procedure is realized to get a less cost design alternative giving the optimum conditions for the system. The proposed model is shown to perform and guarantee a proper solution with a minimum head loss optimization strategy and the main problems included in the three important stages are controlled simultaneously. A more effective decision support system could be developed in the future by paying attention to this model providing consistent, objective and reliable solutions.

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