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The Least Cost Design of Water Distribution Networks Using Water Quality Constraints

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ABSTRACT

Water quality is a very important issue related to people health, but it is not usually considered in water distribution networks (WDNs) design. In this paper, new constraints such as free residual chlorine and the quality performance index are incorporated into the least cost design of water distribution networks. EPANET2 was applied for the hydraulic and quality analysis of water distribution networks. GA was also used to solve the optimization problem. The method was evaluated using a well-known test network. Results showed that inclusion of quality constraints leads to a higher cost. In addition, sensitivity analysis of velocity constraints showed that hydraulic and quality constraints are required for the least cost design of WDNs. On the other hand, the optimum design should satisfy the design criteria at both the start and end of the design period.

Keywords

Quality analysis; Optimization; Genetic Algorithms; Residual chlorine

1. Introduction

Water distribution networks are generally composed of many interconnected pipes, reservoirs, pumps, valves and other hydraulic elements that carry water to demand nodes from the supply sources, with specific pressure levels to provide a good service to consumers. Providing safe drinking water to consumers, free from pathogenic and other undesirable organisms, is the primary goal of all water utilities. The least cost design of water distribution networks has been extensively investigated in many research studies in past decades (Alperovits and Shamir, 1977; Eiger et al., 1994; Vairavamoorthy and Ali, 2005; Broad et al., 2005; Haghighi et al., 2011). Normally pipe diameter is used as the decision variable and hydraulic parameters of nodal pressure and pipe velocity are the most common constraints for optimizing the problem. Besides the hydraulic parameters, quality of water should be maintained within a standard range during the life cycle of any WDN. Quality of water is normally determined by measuring some specific parameters such as residual chlorine, fluoride, water age, etc. However, during the design procedure usually quality constraints have not been considered. Instead, there are several researches about optimizing residual chlorine during the operational period. In these researches residual chlorine is normally considered by sampling from different nodes (Munavalli and Mohan Kumar, 2003; Ostfeld and Salamons, 2006; Tabesh et al., 2011). All the aforesaid studies are often viewed as a single-objective, however multi-objective optimization of WDN is recently growing (Prasad & Park, 2004; Kapelan et al., 2005; Di

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pierro et al., 2009; Kang and Lansey, 2010; Baños et al., 2011).

To optimize the problem, several methods and algorithms have been applied besides linear programming (Alperovits and Shamir, 1977; Hsu et al., 2008) and nonlinear optimization models (Varma et al., 1997). In recent years, direct search methods have been very popular to solve complex problems such as water distribution systems. Genetic Algorithm (Vairavamoorthy & Ali, 2000; Parasad et al., 2003; Samani and Mottaghi, 2006; Weickgenannt et al., 2010), Tabu Search method (Cunha & Sousa, 1999), Simulating Annealing (Lippai et al., 1999; Banos et al., 2009), Ant Colony systems (Maier et al., 2003; Zecchin et al., 2005; Diwold et al., 2010), Particle Swarm Optimization algorithms (Yang and Zhai, 2009; Niu et al., 2011) and Shuffled Frog Leaping algorithm (Eusuff & Lansey, 2003; Chung and Lansey, 2009) are some of the examples for search methods and their applications in optimization of WDNs. Among these methods, GA has been more developed for applying to WDN (Savic and Walters, 1997; Ostfeld and Karpibka, 2005; Blobnesi et al., 2009).

This paper aims to evaluate the effects of quality constraints and pressure dependent analysis on least cost design of WDNs. Based on the standard codes, optimum bounds for residual chlorine are determined and a performance index is introduced in this regard. Then, the minimum cost of water distribution network is obtained using GA considering both hydraulic and quality constraints.

2. Methods

2.1. Optimization Approach

In general, a water quality-hydraulic-based optimization problem aims to minimize the cost, while constraints on the pressures and chlorine concentrations at demand nodes are within certain bounds. The optimal design formulation used in this paper is given by Eqs. (1) to (5) (Alperovits & Shamir, 1977).

$$Min\ Cost = 1.1 * \sum_{i=1}^{NP} L_i D_i^{1.5} \tag{1}$$

$$D \in \{Comercial\ List\} \tag{2}$$

$$H_{\min} \leq H_j \leq H_{\max} \tag{3}$$

$$V_{\min} \leq V_j \leq V_{\max} \tag{4}$$

$$C_{\min} \leq C_j \leq C_{\max} \tag{5}$$

where L_i is the length of pipe i, D_i is diameter of pipe i, H_{min} and H_{max} are the lower and upper limits of operation pressure head in the network, respectively. H_{min} is assumed to be 30 m. V_{min} and V_{max} are the lower and upper limits of flow velocity in network pipes, respectively. C_{min} and C_{max} are the lower and upper limits of chlorine levels, respectively, that may be obtained from the standard codes such as WHO. In this paper, C_{min} and C_{max} are assumed to be 0.2 mg/l and 0.5 mg/l, respectively. H_j and C_j are the head and chlorine concentration in node j, respectively.

GA was applied to solve the optimization problem. Decision variables include pipe diameters that were considered as integer values. Viravamoorthy and Ali (2005) method was applied to obtain the initial population. This methodology improves convergence of the solution. Here, 20% of the initial population was produced randomly, to maintain diversity. Arithmetic crossover and Gaussian mutation (Viravamoorthy and Ali, 2000) were also applied. To optimize this problem with GA, constraints were converted into penalty functions. Penalty functions for pressure head and chlorine concentration are given by Eq. (6):

$$\begin{aligned} PF &= R \times \sum_{j=1}^{NJ} \left[\max\{0, (H_{\min} - H_j), (H_j - H_{\max})\}\right] \\ PF &= \sum_{j=1}^{NJ} R \times \left[\max\{0, (C_{\min} - C_j), (C_j - C_{\max})\}\right] \end{aligned} \tag{6}$$

in which PF is the penalty function, R is the penalty multiplier for pressure head (\$/m) that is obtained by using sensitivity analysis.

The Fitness Function (FF) to each chromosome is given by Eq. (7) that was minimized during optimization:

$$FF = f(D) = \frac{1}{Cost + \sum PF} \tag{7}$$

in which cost is calculated by Eq. (1) and $\sum PF$ is the summation of penalty functions for all constraints.

2.2. Simulation Approach

For Hydraulic analysis of water distribution network, the EPANET2 software (2000) was used. It is an open source program that can be easily integrated with an optimization model.

For quality analysis, the Discrete Volume Element Method (DVEM) was used. This method, originally introduced by Rossman et al. (1993), is a dynamic explicit approach. It is a one-dimensional model which assumes full mixing at nodes and ignores longitudinal dispersion. The algorithm was predicted on a mass balance equation that accounts for both advective transport and reaction kinetics.

In the DVEM, each pipe was divided into a number of volumetric elements. The concentration in each element was determined considering the initial concentration of upstream and downstream nodes after reaction and was transferred into the next element. The nodal concentration was updated assuming full mixing at nodes.

This procedure was repeated at any quality time step until the next hydraulic time step. Normally, quality time steps were much less than the hydraulic ones to consider any short travel time which might occur inside the pipes (Rossman et al., 1993). Discharge and velocity values were constant during a hydraulic time step. During this period, the concentration values in the pipe i, point x and time t, $[C_i(x,t)]$ were determined using Eq. (8):

$$\frac{\partial C_i(x,t)}{\partial t} + u_i \frac{\partial C_i(x,t)}{\partial x} - r\left[C_i(x,t)\right] = 0 \tag{8}$$

where u_i is the mean velocity of water in pipe i and $r\left[C_i(x,t)\right]$ is the reaction rate, which for the first order reaction, is equal to:

$$r(C_i) = \alpha\, C_i \tag{9}$$

where α denotes a coefficient of concentration decay (negative) or growth (positive) rate and is zero for conservative substances, calculated using Eq. (10) (Walski et al., 2003):

$$\alpha = k_b + \frac{k_w k_f}{r_H (k_w + k_f)} \tag{10}$$

in which k_f is the mass transfer coefficient, bulk fluid to the pipe wall and r_H is hydraulic radius of pipeline and k_b and k_w are bulk and wall reaction coefficients, respectively.

EPANET 2 (Rossman, 2000) was used for DDSM and quality analysis and HDSM model of Tabesh and Dolatkhahi (2006) was used for hydraulic and quality analyses. For quality constraints, two different scenarios were considered. In the first stage, values of nodal residual chlorine were bounded between 0.2-0.5 mg/lit (recommended by standard codes such as WHO (2003) and IMPO (2013)). In the second stage, a quality performance index was applied using the penalty curves shown in Fig. 1 for residual chlorine (Coelho, 1996). In Fig. 1, for the optimum range of 0.2-0.5 mg/lit, the performance index was 1 which is excellent. Values of 0.175 and 0.6 mg/lit were considered as good performance, shown by an index of 0.75. Values of 0.15 and 0.7 mg/lit, which show the index of 0.5, were considered acceptable. Residual chlorine values more than 0.8 mg/lit were considered as unacceptable values and ranked as 0.25. Finally, cases of chlorine concentration less than 0.1 mg/lit corresponded to no service, which is completely unacceptable.

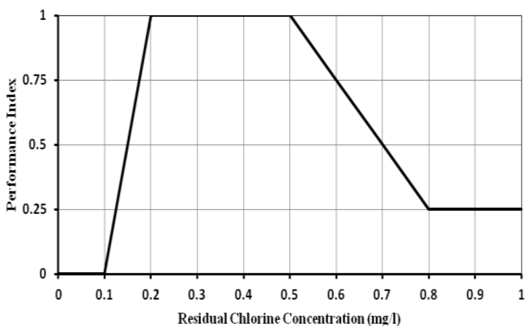

Fig. 1. Penalty curve for residual chlorine (Coelho 1996, Tabesh and Doulatkhahi, 2006)

To generalize the quality performance index of different elements of the entire network, the following equation was used.

$$PI_N = W(PI_j) = \frac{\sum_{j=1}^{NJ} Q_j^{req}.PI_j}{\sum_{j=1}^{NJ} Q_j^{req}} \tag{11}$$

where PI_N is the network performance index, PI_j is the performance index for node j and NJ is the total nodes.

The proposed methodology can be summarized as Fig. 2. A computer code was prepared in Matlab 7.1 to solve the problem. The specifications of GA to solve the optimization problem obtained by sensitivity analysis were as follows: the population size = 50, no. of generation = 250, no. of best solution that is transferred to the next generation without change = 5, probability of crossover (Pc) = 1, probability of mutation (Pm) = 1/8 (Viravamoorthy and Ali, 2005). The penalty value of quality constraint was 1010 and for pressure constraint, it was considered in four ranges. For pressure values more than 30 m, in the range of 20-30 m, in the range of 10-20 m and less than 10 m deviation from the constraint bound, penalty values of 108, 54, 39 and 27 were applied, respectively.

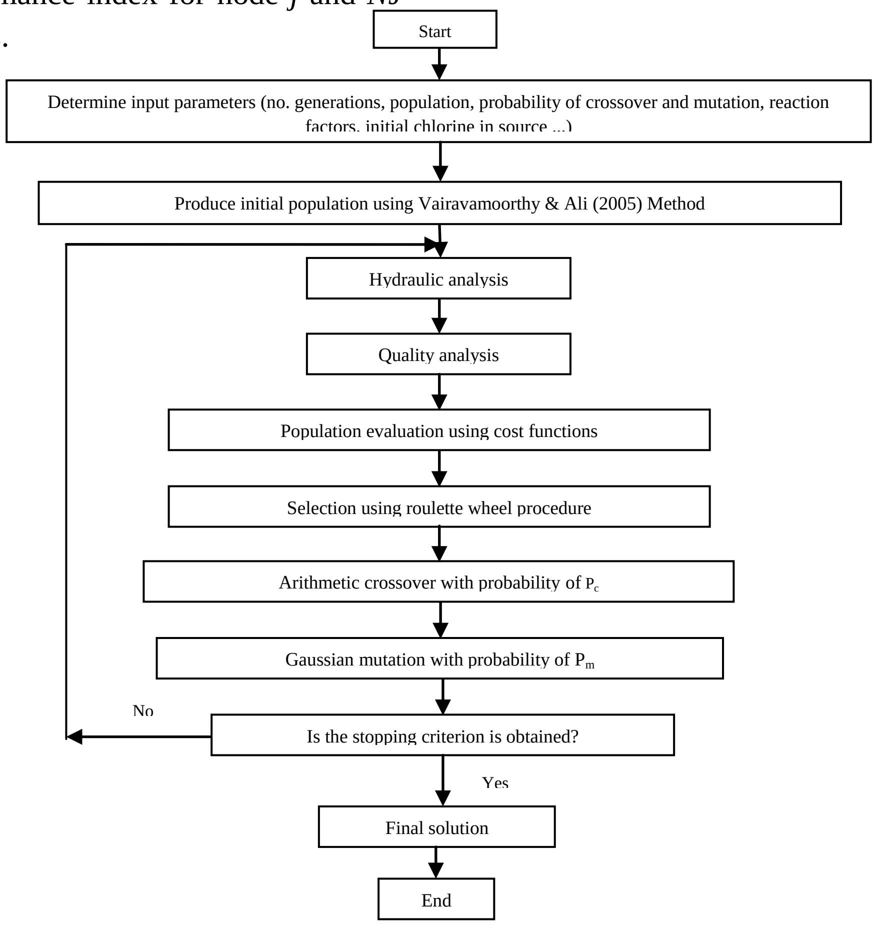

Fig. 2. Flowchart of the proposed methodology

3. Results and Discussion

The well-known two loop network of Alperovits & Shamir (1977) shown in Fig. 3 was considered to evaluate the proposed methodology. Nodal data are presented in Table 1. Pipe lengths were 1000 m and Hazen-Williams coefficient of pipes was 130. Desired head at each node was 30 m. Pipe costs are presented in Table 2.

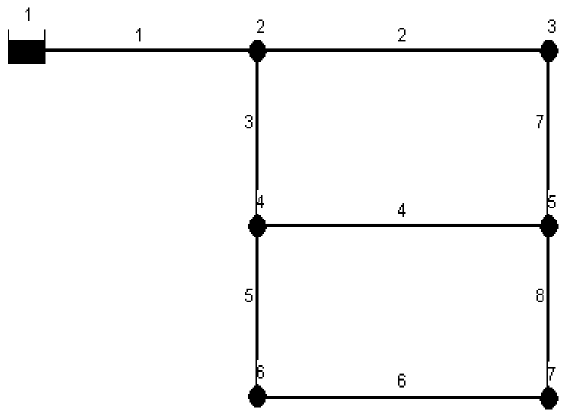

Fig. 3. Layout of the test example (Alperovits & Shamir, 1977).

Table 1. Nodal data for the test example (Alperovits & Shamir, 1977).

Node No.	1	2	3	4	5	6	7
Demand (m^3/hr)	-	100	100	120	270	330	200
Elevation (m)	210	150	160	155	150	165	160

Table 2. Cost of pipes (Alperovits & Shamir 1977).

Diameter (mm)	Cost ($/unit length of pipe)
25.4	2
50.8	5
76.2	8
101.6	11
152.4	16
203.2	23
254	32
304.8	50
355.6	60
406.4	90
457.2	130
508	170
558.8	300
609.6	550

Table 3. Optimum cost and pipe diameter for the test example.

Pipe No.	Pipe diameter (mm)			
	Present study	Savic &Walters (1997)	Abede & Solomatine (1998)	Cunha & Sousa (1999)
1	457.2	457.2	457.2	457.2
2	355.6	254	355.6	254
3	355.6	406.4	355.6	406.4
4	25.4	101.6	25.4	101.6
5	355.6	406.4	355.6	406.4
6	152.4	254	25.4	254
7	355.6	254	355.6	254
8	254	25.4	304.8	25.4
Cost	420	419	424	419

First, the construction cost and pipe diameters were calculated without considering the quality constraints. In Table 3, results are compared with the other available research outputs. It can be seen that the minimum cost obtained in this paper is close to the other research results and the observed differences are because of the various approaches applied.

Then, to show the effects of the quality constraints on optimum design of the WDNs, the effects of bulk reaction coefficient, k_b, and wall reaction coefficient, k_w, were investigated. Cost and PI values were evaluated when these two coefficients varied (between 0-5) and chlorine concentration was within the range of 0.2-0.5 mg/lit. Results are presented in Table 4 for two case; demand at the end of the design period (Case A) and demand at both the start and end of the design period (Case B). Demand at the end of design period was considered because pressure constraint was critical at this time and demand at the start of design period was considered because demand and velocity in pipes were low. Therefore, at the start of design period, chlorine decay was high and conseque-ntly, chlorine concentration was critical. It should be noted that the initial chlorine at source node was 0.5 (mg/lit) and an evaluation was performed after 48 hours, when the residual chlorine became stable in the system. Since the data of minimum demand was not available for this network, therefore one third of the maximum demand was considered as the minimum demand.

It was seen that for some values of these two coefficients with available diameters, the problem does not achieve to the feasible solution in which all quality constraints at each node can be satisfied. Therefore, PI values were less than 100% (underlined data). This situation was more critical in case B when both start and end demand values were considered. It means that when the demands of the start of design period are considered, satisfaction of quality constraints is getting worth. It was observed that the solution was not sensitive to low values of the coefficients and it was going towards infeasible solution for high values of the coefficients. Therefore, it can be concluded that assuming k_b and k_w equal to 2.5 was suitable. It was observed that most of the researchers have used the same value. On the other hand, there was not a fix trend between the increase of two coefficients, k_b and k_w, and the cost because of the confliction between the two pressures and quality constraints. In fact, satisfaction of pressure constraints leads to an increase in the diameters and satisfaction of quality constraints leads to a decrease in the diameters. Table 5 shows the results of the test example network when k_b and k_w coefficients were constant and equal to 2.5 and chlorine concentration was within the range of 0.2 to 0.5 mg/lit. For evaluation of the quality constraints, the demand values at the beginning of the WDN life cycle was considered besides the design demand, because velocities were lower and therefore, residual chlorine at nodes were reduced. It means that in contrast to pressure, the start of the WDN life period is more critical for quality constraints.

Table 4. Variations of cost and *PI* against k_w and k_b.

Case A (Cost *103 $)			k_w					
			0	1	2	3	4	5
k_b	0	Cost	429	442	442	452	424	434
		PI	1	1	1	1	1	1
	1		420	442	442	429	424	505
		PI	1	1	1	1	1	1
	2	Cost	442	420	442	424	498	572
		PI	1	1	1	1	1	1
	3	Cost	442	442	442	505	534	624
		PI	1	1	1	1	1	0.980
	4	Cost	442	442	420	505	554	582
		PI	1	1	1	1	0.999	0.960
	5	Cost	420	442	424	514	544	544
		PI	1	1	1	1	0.982	0.942
Case B (Cost*103$)			k_w					
			0	1	2	3	4	5
k_b	0	Cost	420	427	754	2302	2638	2868
		PI	1	1	1	1	0.985	0.982
	1	Cost	423	536	643	994	2047	2533
		PI	1	1	0.931	0.888	0.860	0.843
	2	Cost	442	553	466	434	567	844
		PI	1	1	0.891	0.789	0.700	0.695
	3	Cost	442	502	436	436	434	464
		PI	1	0.999	0.787	0.686	0.612	0.567
	4	Cost	438	553	436	436	436	436
		PI	1	0.918	0.744	0.609	0.544	0.502
	5	Cost	438	544	436	436	436	436
		PI	1	0.889	0.650	0.545	0.483	0.444

Table 5. Pipe diameter, cost of design and total PI for the test example (k_b = k_w =2.5)

	Optimum Diameter (mm)	
Pipe No.	Case A	Case B
1	457.2	508
2	355.6	355.6
3	355.6	355.6
4	152.4	25.4
5	355.6	304.8
6	101.6	25.4
7	355.6	355.6
8	254	254
Cost	429	436
PIN	1	0.7997

Comparison between Tables 3 and 5 shows that considering quality constraints leads to an increase in the design cost, so that the costs of design with quality constraints (429) was higher than the costs of design without quality constraints (420). Considering quality constraints, when demands at both start and end of the design period were evaluated, design cost increased and performance value decreased because of the confliction between the pressure and quality constraints. To consider this problem, optimization results are illustrated in Table 6 for cases A and B and different sets of demands.

The results showed that incorporation of quality constraints and different set of demands leads to an increase in design cost. On the other hand, to satisfy the quality constraints at both start and end of the design period, higher cost and lower PI were obtained from the network design. It can be stated that to satisfy pressure constraint at nodes, pipe diameters increased by the optimization procedure. In contrast, to satisfy the quality constraint, the model increased the velocity of pipes that leads to a decrease in pipe diameters.

It was seen that when the design demand was considered in cases 2 and 3 in this network, diameter of pipes 4 and 6 increased and decreased, respectively in comparison to case 1. When the demand at the start of design period was considered, diameter of pipes 3 and 6 increased and diameter of pipe 8 decreased in case 2.

Table 6. Optimum diameters (mm) and costs ($US)

No.	Case A			Case B		
	Case 1	Case 2	Case 3	Case 3	Case 2	Case 1
1	457.2	457.2	457.2	457.2	457.2	↑508
2	355.6	355.6	355.6	355.6	355.6	355.6
3	355.6	355.6	355.6	355.6	↑406.4	355.6
4	25.4	↑203.2	↑144	25.4	25.4	25.4
5	355.6	355.6	355.6	355.6	355.6	↓304.8
6	144	↓25.4	↓144	144	↑203.2	↓25.4
7	355.6	355.6	355.6	355.6	304.8	355.6
8	254	254	254	254	↓203.2	254
Cost	420	427	429	436	420	436

Case 1: Only pressure constraint (30m≤H_j)
Case 2: Pressure and quality cons. (0.2≤C_j ≤0.5, 30≤ H_j, k_b = 2.5)
Case 3: Pressure and quality cons. (0.2≤ C_j ≤0.5, 30≤ H_j, k_b = k_w=2.5)
↑ = Diameter increase, ↓ = Diameter decrease

When demand at the start of design period was used, besides the design demand, the feasible solution which satisfies all constraints was not obtained in case 3. In this case, pressure at node 8 was 29.98 m and residual chlorines at the start of the design period in nodes 7 and 8 were 0.1814 and 0.1184 mg/lit, respectively. In this situation, the problem cannot reach a feasible solution by increasing the diameter of further pipes and increasing the diameter of the closer pipes to the reservoir. In this example, the proposed algorithm tried to reach a solution with minimum cost, considering penalty functions by increasing diameter of pipe 1 and decreasing diameter of pipes 4 and 6. This represents the criticality of considering both coefficients of k_b and k_w and demand at both start and end of the design period.

In the next step, quality performance index for residual chlorine was applied as the quality constraint. A range of nodal PIs was investigated. The results are illustrated in Table 7. Two different scenarios with fixed and variable chlorine injection into the reservoir were considered. It was seen that when PI varied from 0.3 to 1 in case C (constant injection into the reservoir), PI_N increased. However, PI in node 7 (the farthest node) decreased. On the other hand, in case D (variable injection based on the penalty curve) both PI_N and PI_7 first increased and then decreased. In comparison to case C, both PI_N and PI_7 were higher in case D.

Table 7. Variations of cost, PI_N and PI_7 considering different bounds for nodal PI

Nodal PI ≥	C) Chlorine in the reservoir is 0.5 (mg/lit)			D) Chlorine in the reservoir is based on the penalty curve			
	PI_N	PI_7	Cost	Initial Chlorine	PI_N	PI_7	Cost
1.00	0.812	0.198	386	0.50	0.812	0.198	386
0.90	0.812	0.198	386	0.54	0.855	0.305	584
0.80	0.799	0.202	431	0.58	0.852	0.494	454
0.70	0.773	0.224	754	0.62	0.853	0.675	514
0.60	0.755	0.285	419	0.66	0.925	0.604	528
0.50	0.737	0.299	507	0.70	0.927	0.659	424
0.40	0.701	0.404	785	0.74	0.881	0.445	420
0.30	0.699	0.302	562	0.78	0.894	0.554	429

* Based on the penalty curve in Fig. 1
k_b=k_w=2.5 , Node 7 is the farthest node to the reservoir.

In another step, to investigate the substitution possibility of the velocity constraint with the quality constraint, Eq. (4) was replaced with the following relation.

$$A \le v_i \le 2.5 \quad (m/s)$$
$$(A = 0,0.2,0.4,0.6,0.8,1.0,1.2,1.6,2) \qquad (12)$$

Values of design cost and quality performance index at the start and end of the design period for two cases of A and B are calculated and presented in Tables 8 to 10.

It was observed that performance indices of all the nodes and the whole network were satisfied at the end of the design period. This is shown in Figs. 4 and 5 for node 7 and the whole network that make known all of the performance indices were higher than 0.954. The reason is that nodal demands were higher at the end of the design period, therefore pipe velocity was higher, chlorine decay was lower and residual chlorine was consequently higher. In contrast, Table 9 and Fig. 6 show that for the start period, PI values of nodes 5, 6, 7 and the whole network were not satisfied. The reason is that nodal demands of case B were lower than case A. Therefore, higher decay and lower residual chlorine occurred. A fix trend was not observed between the velocity constraint and the quality performance index. Because of the looped network, decrease in the diameter of one or few pipes will increase the velocity in pipes; however, PI in one or few nodes may decrease.

Table 8. Effects of the velocity limit on nodal PI and pipe velocity at the end of design period (case A).

		Lower Bound of Velocity (m/s)									
		0	0.1	0.2	0.4	0.6	0.8	1	1.2	1.6	2
	PI_N	0.992	1	0.992	1	1	1	0.993	1	1	1
	PI_2	1	1	1	1	1	1	1	1	1	1
	PI_3	1	1	1	1	1	1	1	1	1	1
	PI_4	1	1	1	1	1	1	1	1	1	1
	PI_5	1	1	1	1	1	1	1	1	1	1
	PI_6	1	1	1	1	1	1	0.977	1	1	1
PI	PI7	0.957	1	0.957	1	1	1	1	1	1	1
Cost ($)		442000	438000	442000	441000	429000	425000	452000	490000	483000	411000

Table 9. Effects of the velocity limit on nodal PI and pipe velocity at the end of design period (case B).

		Lower Bound of Velocity (m/s)									
		0	0.1	0.2	0.4	0.6	0.8	1.0	1.2	1.6	2
	PI_N	0.992	0.992	1	0.992	1	1	1	1	1	1
	PI_2	1	1	1	1	1	1	1	1	1	1
	PI_3	1	1	1	1	1	1	1	1	1	1
PI	PI_4	1	1	1	1	1	1	1	1	1	1
	PI_5	1	1	1	1	1	1	1	1	1	1
	PI_6	1	1	1	1	1	1	1	1	1	1
	PI7	0.957	0.957	1	0.954	1	1	1	1	1	1
Cost ($)		442000	442000	447000	458000	427000	427000	470000	422000	433000	392000

Table10. Effects of the velocity limit on nodal PI and pipe velocity at the start of design period (case B)

		Lower Bound of Velocity (m/s)									
		0	0.1	0.2	0.4	0.6	0.8	1	1.2	1.6	2
	PI_N	0.740	0.739	0.763	0.538	0.732	0.732	0.550	0.747	0.581	0.581
	PI_2	1	1	1	1	1	1	1	1	1	1
	PI_3	1	1	1	1	1	1	1	1	1	1
PI	PI_4	1	1	1	1	1	1	1	1	1	1
	PI_5	0.660	0.66	0.760	0.853	0.631	0.631	0.893	0.691	0.880	0.922
	PI_6	1	1	0.997	0	1	1	0	1	0.111	0.049
	PI_N	0	0	0	0.263	0	0	0.274	0	0.283	0.327
Cost ($)		442000	442000	447000	458000	427000	427000	470000	422000	433000	392000

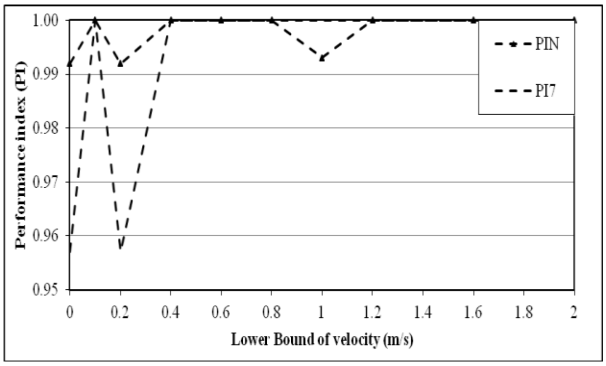

Fig. 4. Performance index versus lower bound velocity at the end of design period (case A).

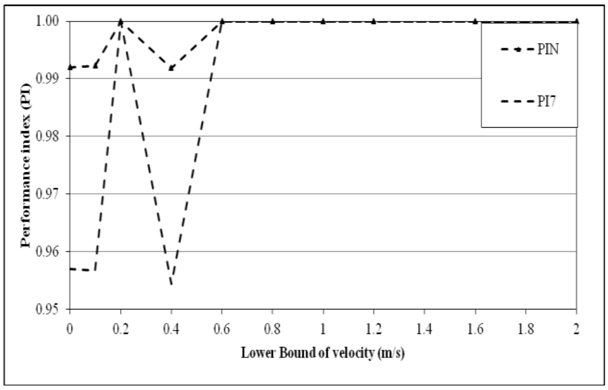

Fig. 5. Performance index versus lower bound velocity at the end of the design period in node 7 and the whole network (case B).

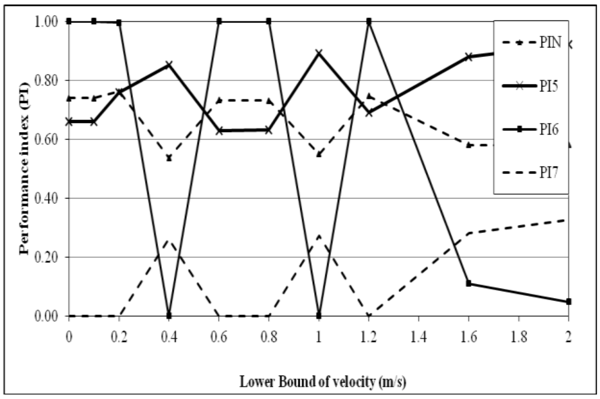

Fig. 6. Performance index versus lower bound velocity at the start of design period in some nodes and the whole network (case B).

In order to further describe the issue assume that the two loop network has a pipe diameter of 600 mm, a bulk reaction coefficient of 2.5 and a chlorine concentration of reservoir of 0.5 (Fig. 7a). Diameter of pipe 3 was changed to 500 mm (Fig. 7b) and 100 mm (Fig. 7c). Velocities in pipes and chlorine concentration in nods are shown in Fig. 7.

Based on Fig. 7b, by decreasing diameter of pipe 3 to 500 mm, velocity in all pipes except pipes 4 and 5, and chlorine concentration in all the nodes increased. On the other hand, performance indices in all nodes except node 6 increased. In Fig. 7c, by decreasing diameter of pipe 3 to 100 mm, velocity in all pipes except pipes 3 and 5 excessively increased and chlorine concentration in all nodes except node 6 increased. Therefore, PI values of node 6 and the whole network decreased.

Results of an investigation showed that any change in diameter of one pipe will directly influence the other pipes. However, in Figs. 7a to 7c just downstream nodes were affected. Therefore, it can be concluded that consideration of just hydraulic constraints of velocity and pressure cannot guarantee the satisfaction of quality constraints. However, in this example consideration of pressure and quality constraints can lead to satisfaction of velocity constraints.

Consideration of design cost with lower bound of velocity in Figs. 8 to 10 showed that there is no fix trend between alteration of lower bound velocity and design cost. Because in the looped network, a decrease in diameter of some pipes will increase the diameter of few pipes. However, constraints on the pressure and chlorine concentrations at demand nodes are within the certain bounds.

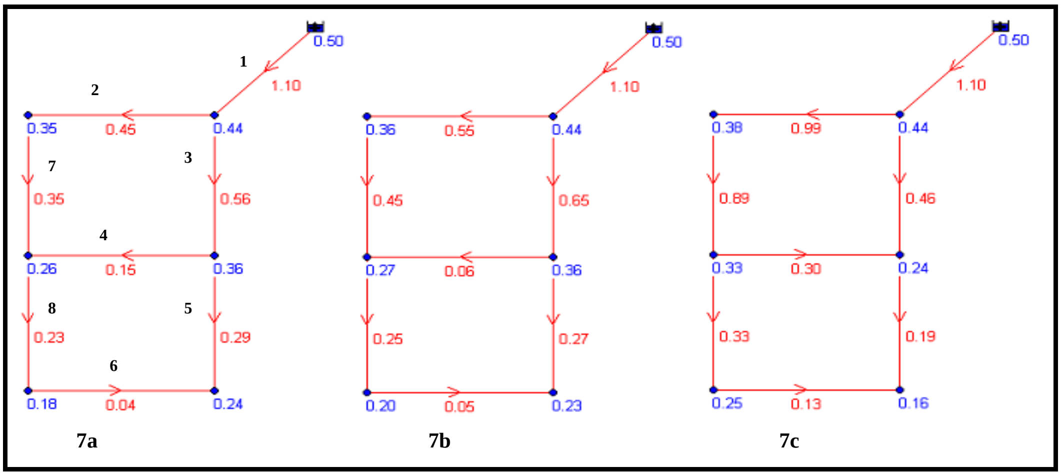

Fig. 7. Variations of velocity and chlorine concentration with pipe diameter.

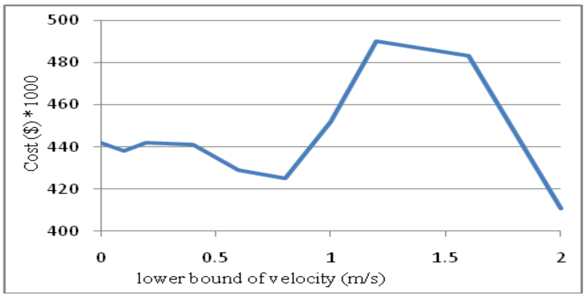

Fig. 8. Design cost versus lower bound velocity at the end of design period (case A).

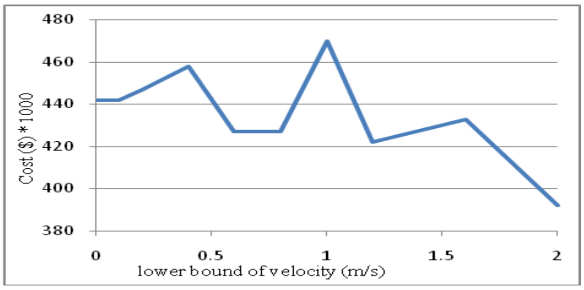

Fig. 9. Design cost versus lower bound velocity at the end of design period (case B).

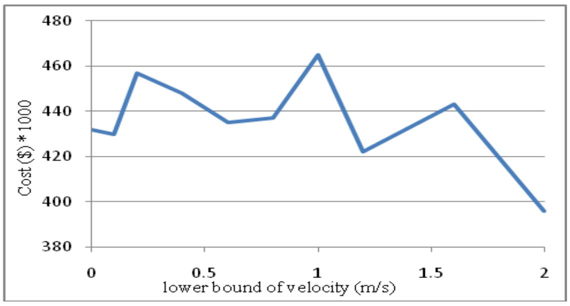

Fig. 10. Design cost versus lower bound velocity at the start of design period (case B).

4. Conclusions

This paper developed an investigation to assess the effects of considering quality constraints on the least cost design of water distribution systems. It was observed that consideration of quality constraints is necessary for optimum design of WDN and adding quality constraints to the optimization problem leads to higher design cost. In addition, consideration of both coefficients of k_b and k_w together with the start and end of design period are necessary when quality constraints are applied. However, selection of suitable k_b and k_w values is important because the solution is not sensitive to low values of these coefficients and it is going towards infeasible solution for high values of the coefficients. Therefore, considering the start and end of the design period leads to higher costs. In addition, conventional constraints of pressure and velocity cannot guarantee satisfaction of residual chlorine within the standard limits. It cannot be concluded that a fixed trend exists between the velocity constraint and the quality performance index. Because in the looped network, a decrease in diameter of one or few pipes will increase the velocity in pipes.

However, PI of one or few nodes may decrease. In addition, the best performance index for residual chlorine was obtained when variable injection was considered in the reservoir.

References

Alperovits, E., and Shamir, U., (1977), Design of optimal water distribution systems, Water Resources Planning and Management. 13(6), 885-900.

Banos, R., Gil, C., Reca, J., and Martínez, J., (2009), Implementation of scatter search for multi objective optimization: a comparative study. Computational Optimization and Applications, Springer, 42, 421–441.

Banos, R., Reca, J., Martínez, J., Gil, C., and Márquez, A.L., (2011), Resilience indexes for water distribution network design: A performance analysis under demand uncertainty. Water Resources Management, 25, 2351-2366.

Broad, D.R., Dandy, G.C., and Maier, H.R., (2005), Water distribution system optimization using metamodels. Water Resources Planning and Management, 131(3), 172-180.

Bolognesi, A., Bragalli, C., Marchi, A., and Artina, S., (2010), Genetic Heritage evolution by stochastic transmission in the optimal design of water distribution networks. Advances in Engineering Software, 41(2), 792-801.

Chung, G. and Lansey, K. (2009), Application of the Shuffled Frog Leaping algorithm for the optimization of a general large-scale water supply system. Water Resources Management, 23(4), 797-823.

Coelho, S.T., (1996), Performance assessment in water supply and distribution. Ph.D. Thesis, Civil & Offshore Engineering Department, Heriot-Watt University, Edinburg, UK.

Cunha, M.C., and Sousa, J., (1999), Water distribution network design optimization: Simulated Annealing approach. Water Resources Planning and Management, 125(4), 215-221.

Di pierro, F., Khu, S.T., Savic, D., and Berardi, L., (2009), Efficient multi-objective optimal design of water distribution networks on a budget of simulations using hybrid algorithms. Environmental Modeling & Software, 24, 202-213.

Diwold, C., Ruhnks, T., and Midendrof, M., (2010), Sensor placement in water networks using a population based ant colony optimization algorithms. Springer-Verlag, ICCCI2010, 3, 426-437.

Eusuff, M. M., and Lansey, K. E., (2003), Optimization of water distribution network design using the shuffled frog leaping algorithm. Water Resources Planning and Management, 129(3), 210-225.

Eiger, G., Shamir, U., and Ben-Tal, A., (1994), Optimal design of water distribution networks. Water Resources Planning and Management, 30(9), 2637-2646.

Haghighi, A., Samani, H., and Samani, Z., (2011), GA-ILP method for optimization of water distribution networks. Water Resources Management, 25, 1791–1808.

Hsu, N.Sh., Cheng, W.C., Cheng, W.M., Wei, C.C., and Yeh, W.G., (2008), Optimization and capacity expansion of a water distribution system. Advances in Water Resources, 31(2), 776–786.

Iran Management and Planning Organization (IMPO). (2013), Standard code for design of urban and rural water distribution systems, Iran Management and Planning Organization and Ministry of Energy, Tehran, Iran.

Kapelan, Z.S., Savic, D.A., and Walters, G.A., (2005), Multiobjective design of water distribution systems under uncertainty. Water Resources Research, 41(11), W11407.1-W11407.15.

Kang, D., and Lansey, K., (2010), Optimal meter placement for water distribution system State estimation. Water Resources Planning and Management, ASCE, 136(3), 337-347.

Lippai, I. Heaney, J.P., and Laguna M., (1999), Robust water system design with commercial intelligent search optimizers. Journal of Computing in Civil Engineering, 13(3), 135-143.

Maier, H.R., Simpson, A.A., Zecchin, A.C., Foong, W.K., Phang, K.Y., Seah, H.Y., and Tan, C.L., (2003), Ant colony optimization for design of water distribution systems. Water Resources Planning and Management, 129(3), 200-209.

Munavalli, G.R., and Mohan Kumar, M.S., (2003), Optimal scheduling of multiple chlorine sources in water distribution systems. Water Resources Planning and Management, 129(6), 493-504.

Niu, Z.G., Wang, Y.F., Zhang, T., and Zhang, H.W., (2011), Calibration of friction resistance coefficient and calculation method of leakage localization of water supply network. Journal of Tianjin University Science and Technology, 44(4), 364-368.

Ostfeld, A., and Karpibka, A., (2005), A GA-LP approach to water distribution systems optimal design. World Water and Environmental Resources Congress 2005, Anchorage, Alaska, United States, May 15-19, pp. 1-6. doi: 10.1061/40792(173)611.

Ostfeld, A., and Salamons, E., (2006), Conjunctive optimal scheduling of pumping and booster chlorine injections in water distribution systems. Engineering Optimization, 38(3), 337–352.

Prasad, T.D., Hang, S.H., and Park, N.S., (2003), Reliability based design of distribution networks using multi-objective Genetic Algorithms. KSCE Journal of Civil Engineering, 3(7), 351-361.

Prasad, T.D., and Park, N.S., (2004), Multi-objective genetic algorithms for design of water distribution networks. Water Resources Planning and Management, ASCE, 130(1), 73-82.

Rossman L.A., Boulos P.F., and Altman T., (1993), Discrete volume element method for network water-quality models. Water Resources Planning and Management, 119(5), 505-517.

Rossman, L.A., (2000), EPANET2: user's manual, U.S. Environmental Protection Agency, Risk Reduction Engineering Laboratory, Cincinnati, Ohio, USA.

Savic, D.A., and Walters G.A., (1997), Genetic algorithms for least-cost design of water distribution networks. Water Resources Planning and Management, 123(2), 67-77.

Tabesh, M., and Doulatkhahi A., (2006), Effects of pressure dependent analysis on quality performance assessment of water distribution networks. Iranian Journal of Science and Technology, Transaction B: Engineering, 30(B1), 119-128.

Tabesh, M., Azadi, B., and Roozbahani, A., (2011), Quality management of water distribution networks by optimizing dosage and location of chlorine injection. International Journal of Environmental Research (IJER), 5(2), 321-332.

Vairavamoorthy, K., and Ali, M., (2000), Optimal design of water distribution systems using genetic algorithms. Computer-Aided Civil and Infrastructure Engineering, 15(2), 374-382.

Vairavamoorthy, K., and Ali, M., (2005), Pipe index vector: a method to improve genetic-algorithm-based pipe optimization. Journal of Hydraulic Engineering© ASCE. 131(12), 1117-1125.

Varma, K., Narasimhan, S. and Bhallamudi, S.M., (1997), Optimal design of water distribution systems using NLP method. Journal of Environmental Engineering, ASCE, 123(4) 381-388.

Walski, T.M., Chase, D.V., Savic, D.A., Grayman, W., Beckwith, S., and Koelle, E., (2003), Advanced water distribution modeling and management. Bentley Institute Press.

Weickgenannt, M., Kapelan, Z., Blokker, M., Savic, D.A., and Sawodny, O., (2010), Optimization of sensor locations for contaminant detection in water distribution networks. IEEE International Conference on Control Applications, 2160-2165.

Yang, K., and Zhai, J., (2009), Particle Swarm Optimization algorithms for optimal scheduling of water supply systems. Second International Symposium on Computational Intelligence and Design, IEEE, 509-512.

Zecchin, C.Z., Simpson, A.R., Maier, H.R., and Leonard M., (2005), Application of two ant colony optimization algorithms to water distribution system optimization. Mathematical and Computer Modeling, 44, 451-468.

Samani, H.M.V., and Mottaghi, A., (2006), Optimization of water distribution networks using integer linear programming. Journal of Hydraulic Engineering, ASCE, 132(5), 501–509.