EFFECTS OF PRESSURE DEPENDENT ANALYSIS ON QUALITY PERFORMANCE ASSESSMENT OF WATER DISTRIBUTION NETWORKS[*]

M. TABESH[**] AND A. DOLATKHAHI

School of Civil Engineering & Technical Pardis, University of Tehran, P.O. Box 11365-6543, Tehran, I. R. of Iran
E-mail: mtabesh@ut.ac.ir

Abstract– Water network performance is defined as the ability to deliver a required quantity of water under sufficient pressure and an acceptable level of quality. A sound performance indicator is a powerful tool for more efficient management of water systems. This paper introduces a methodology for performance assessment of water distribution networks based on quality parameters (such as residual chlorine, water age, etc.) and the head driven simulation method (HDSM). For hydraulic analysis of water networks a pressure dependent simulation model is used. This model is able to predict the hydraulic behavior of the system more realistically, especially during abnormal and critical conditions (e.g., outage of pumps and reservoirs, pipe breaks, leakage, excess demands, etc.). Also, a discrete-volume element method (DVEM) is applied for the analysis of water quality parameters.

In the next step, using penalty curves based on the standard codes for quality parameters, the quality performance of the system is assessed. By evaluating a test network, the application of the new methodology is presented. The results are also compared with the widely used water quality simulator of EPANET 2 software, which uses the demand-driven simulation method (DDSM) as its hydraulic simulation engine. The DDSM models consider fixed demands regardless of nodal pressure variations. Consideration of HDSM leads to different pipe velocities, and therefore, different values for quality parameters. The results showed that the introduced procedure can help to assess the performance of quality parameters in water distribution networks more realistically than the existing demand-driven simulation based models.

Keywords – Water networks, head-driven simulation method, performance index, water quality, residual chlorine, water age

1. INTRODUCTION

Performance of a water distribution network can be defined as its ability to deliver a required quantity of water under sufficient pressure and an acceptable level of quality during different normal and abnormal operational situations. Water distribution network performance can be assessd from different points of view including water quality parameters (e.g., residual chlorine, water age, etc.). In the past, several researchers have studied this concept under the topics of Level of Service [1], Reliability [2-4], and Performance Index [5]. The latter introduced a performance assessment index to evaluate the performance of quality parameters in water distribution systems. However, this model was based on the results of a demand-driven simulation hydraulic model.

As hydraulic network analysis can produce good estimates of the network's hydraulic variables, it is possible to obtain a sufficiently accurate picture of the behavior of certain categories of water quality parameters by means of mathematical modeling. Various water quality models have been presented in distribution systems. These models have used both steady state [6-8] and dynamic formulations [5, 9-13].

Dynamic water quality models can be classified spatially as either a Eulerian or Lagrangian type. The Eulerian approaches move water between fixed grid points or volume segments in pipes as time is

---

advanced in uniform increments. These are divided into two methods: the Finite-Difference Method (FDM) and the Discrete-Volume Method (DVM). The Lagrangian methods update conditions in variable sized segments of pipes at either uniform time increments (Time Driven Method, TDM) or only at times when a new segment reaches a downstream pipe junction (Event Driven Method, EDM). The Lagrangian methods are more efficient for simulating chemical transport. For modeling water age, TDM is the most efficient method, while the Eulerian methods are more memory efficient [14].

To analyze the quality parameters, at first a hydraulic simulation model is required. Unfortunately, there are no satisfactory methods for calculating the quantity of flow actually delivered by a water distribution system with less than satisfactory pressure. To date, all the well-established commercial software for the analysis of water distribution systems are based on the DDSM, which assumes that values of nodal demands are fixed and known in advance. Although this may be reasonable under normal operating conditions, algorithms based on it cannot satisfy systems in which demands are less than the required values at some nodes.

On the other hand, considering a nodal pressure – outflow relationship, HDSM-based models are able to evaluate the hydraulic parameters of nodal heads and velocity in pipes more realistically than DDSM-based models, especially during abnormal situations [15, 16]. All the existing software which are able to analyze the quality parameters, e.g., EPANET 2.0 [5, 17], use a DDSM-based hydraulic analyzer. Since the hydraulic parameters are different in these two types of analyses, more studies are needed to evaluate the HDSM-based quality analysis.

In this paper the algorithm of water distribution networks quality analysis is linked with a HDSM model. The discrete volume method is used for quality analysis because of its simplicity [9]. Some penalty curves are then used to evaluate the performance of quality parameters in network elements (nodes). Thereafter, network performance is assessed for both types of hydraulic simulation methods.

2. METHODOLOGY

a) Hydraulic analysis

The effects of pressure variations on nodal outflows is identified by a relationship between nodal outflow and pressure in the HDSM. In this paper, the following relationship is used to evaluate the pressure dependency of outflows [2]

$$Q_j^{avl} = Q_j^{req} \; ; \; if \;\; H_j \geq H_j^{des}$$

$$Q_j^{avl} = Q_j^{req} (\frac{H_j - H_j^{\min}}{H_j^{des} - H_j^{\min}})^{(\frac{1}{n})} ; \; if \; H_j^{\min} \prec H_j \prec H_j^{des} \quad (1)$$

$$Q_j^{avl} = 0 \; ; \; if \;\; H_j \leq H_j^{\min}$$

where Q_j^{avl} and Q_j^{req} are the available outflow and required demand at node j, respectively, H_j^{des} is the desired head to satisfy the demand, H_j is the available head, H_j^{min} is the minimum head at node j, and n is an exponent, usually between 1.5 and 2. Furthermore, the HDSM program is capable of evaluating the leakage value at each pipe as shown below [18]

$$Q_{L,ij} = C_l L_{ij} \left[0.5 \left(H_i - GL_i + H_j - GL_j \right) \right]^{1.18} \quad (2)$$

in which $Q_{L,ij}$ is the leakage discharge at pipe ij, L_{ij} is the length of pipe ij, C_l is the network leak coefficient and GL_i is the ground level at node i.

By incorporating Eqs. (1) and (2) and using the Hazen-William equation, the nodal continuity equation at node j, F_j, is written as follows [15, 16]:

$$F_j = \sum_{i \in \{j\}}^{\beta j} (\frac{|H_i - H_j|}{K_{ij}})^{0.54} Sgn(H_i - H_j) - Q_j^{req} (\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}})^{0.5}$$
$$-0.5 \sum_{i \in \{j\}}^{\beta j} C_l L_{ij} (\frac{H_i - GL_i + H_j - GL_j}{2})^{1.18} = 0 \tag{3}$$

in which K_{ij} is the friction factor in pipe ij, {j} accounts for all pipes connected to node j and sgn represents the sign of flow direction in pipe ij.

Because demand varies during a 24 hour period affecting water quality parameters, an Extended Period Simulation is required. In this paper a Direct Method [19] is used in which the reservoir variation at each time is identified by the following equation:

$$H_{rs}(t + \Delta t) = H_{rs}(t) + \frac{\Delta V_{rs}(t, t + \Delta t)}{f'_{rs}[H_{rs}(t)]} \tag{4}$$

where H_{rs}, ΔV_{rs} and $f'_{rs}(H_{rs})$ are the head, variation of volume at time interval $(t, t + \Delta t)$ and cross section area of the reservoir, respectively. More details about the HDSM formulations and algorithms can be found in [15].

b) Quality Analysis

For quality analysis the discrete-volume element method (DVEM) is used. This method, first introduced by Rossman *et al*. [9], is a dynamic explicit approach. It is a one dimensional model which assumes full mixing at nodes and ignores longitudinal dispersion. The algorithm is predicted on a mass balance equation that accounts for both advective transport and reaction kinetics.

In the DVEM, each pipe is divided into a number of volumetric elements and the concentration in each element is determined considering the initial concentration of upstream and downstream nodes after reaction and transfer to the next element. The nodal concentration is updated assuming full mixing at nodes, and volume and concentration from incoming pipes.

This procedure is repeated in any quality time step until the next hydraulic time step. Normally quality time steps are much less than the hydraulic ones considering any short travel time which might occurr inside pipes [9]. Discharge and velocity values are constant during a hydraulic time step. In this period the concentration value in pipe i, point x and time t, [$C_i(x,t)$], is determined by the following differential equation:

$$\frac{\partial C_i(x,t)}{\partial t} + u_i \frac{\partial C_i(x,t)}{\partial x} - R[C_i(x,t)] = 0 \tag{5}$$

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

$$R(C_i) = \alpha \ C_i \tag{6}$$

where α denotes a coefficient of concentration decay (negative) or growth (positive) rate and is zero for conservative substances. The next equation is obtained with the substitution of Eq. (5) into Eq. (6):

$$C_i(x, t + \tau) = C_i(x - u_i\tau, t) \ e^{\alpha\tau} \quad \forall \tau \ \leq \ \frac{x}{u_i} \tag{7}$$

in which τ is the quality analysis time step. This equation indicates an exponential kinetic concentration change for advection of any distribution of substance concentration in pipe i at time t for a distance of $u_i \tau$ at time interval τ. Assuming the full mixing procedure and neglecting detention time at nodes, the concentration at each connection i is expressed as:

$$C_i(i,j) = C_k(t) = \frac{\sum\limits_{j \in \{k\}}^{\beta k} Q_j C_j(L_j, t)}{\sum\limits_{j \in \{k\}}^{\beta k} Q_j} \quad (8)$$

where C_k is concentration at node k and $\{k\}$ accounts for all pipes connected to node k. Q_j and L_j are the discharge and length of pipe j, respectively. Furthermore, for a storage tank with an incoming pipe i the concentration is as follows:

$$C_T(t+\tau) = \frac{1}{V_T(t) + Q_i \tau}\Big[C_i(L_i, t)Q_i\tau + V_T(t)C_T(t)\Big] \quad (9)$$

in which C_T and V_T are the fully mixed concentration and volume of the tank, respectively [9]. It is also assumed that an outgoing pipe j carries the fully mixed tank concentration, i.e.:

$$C_j(0, t+\tau) = C_T(t) \quad (10)$$

By inclusion of HDSM and DVEM, a pressure dependent quality analyzer is developed [20, 21].

c) Quality Performance Index

For performance assessment, the penalty curves shown in Figs. 1 and 2 are applied for residual chlorine and water age, respectively [5]. In Fig. 1, for an optimum range of 0.2-0.5 mg/l (recommended by WHO) the performance index is 1, which means excellent. Values of 0.175 and 0.6 mg/l are considered as "good performance", shown by an index of 0.75. Values of 0.15 and 0.7 mg/l which show the index of 0.5 are thought to be "acceptable". Residual chlorine values more than 0.8 mg/l are considered "unacceptable" and ranked as 0.25. Finally, any situation with less than 0.1 mg/l chlorine concentration corresponds to "no service", which is completely unacceptable.

Figure 2 illustrates that any travel time below the concentration time limit (Tl) is considered as an excellent performance and graded as 1. From (Tl) to maximum time (Tm) the performance is acceptable, and above this it is totally unacceptable. Tl and Tm are considered as 6 and 10 hours respectively, in this paper.
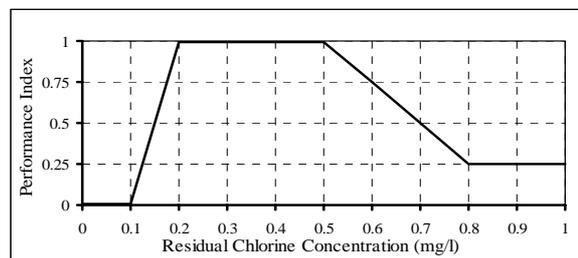

Fig. 1. Penalty curve for residual chlorine

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

$$PI = W(PI_j) = \frac{\sum\limits_{j=1}^{NJ} Q_j^{req} . PI_j}{\sum\limits_{j=1}^{NJ} Q_j^{req}} \quad (11)$$

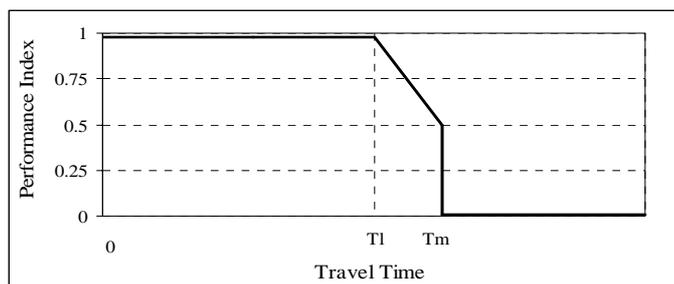where PI is the network performance index, PI_j is the performance index for node j and NJ is the total nodes.

Fig. 2. Penalty curve for water age

3. APPRAISAL

To evaluate the proposed methodology the test network of Fig. 3 (taken from [17]) is considered. The nodal and pipe data are shown in Tables 1 and 2. The network is analyzed for a period of 24 hours with 1 hr. time intervals for hydraulic simulation and 5 minute time intervals for quality analysis. Demand varies over a 24 hour period and the average of the demand pattern is 1. The characteristic pump equation is $H_P = -0.002837Q_p^2 + 101.6$, tank diameter is 15.39 m with an initial water level of 36.576 m. Also $\alpha = -1 \ day^{-1}$ and $C_l = 1 \times 10^{-6}$.
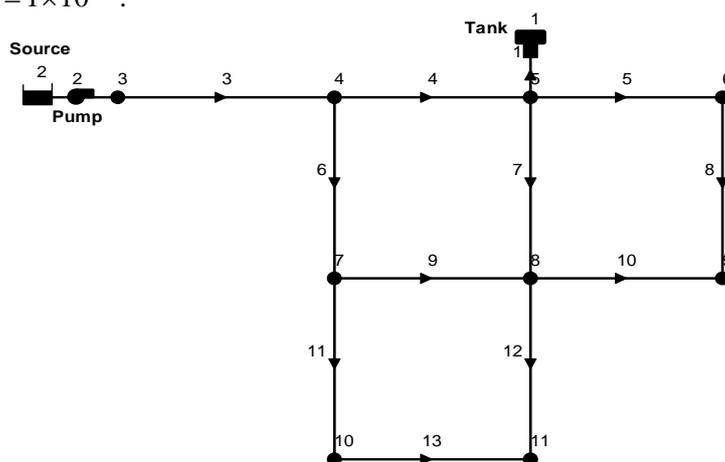

Fig. 3. Layout of the test network

Table 1. Nodal data

Node No.	El. (m)	Initial head (m)	Q_j^{req} (l/s)	Initial Chlo. Conc. (mg/l)
1 (Tank)	289.6	295.6	0	1
2 (Res.)	243.8	243.8	0	1
3	216.4	306.1	0	0.5
4	216.4	300.2	9.5	0.5
5	213.4	295.6	9.5	0.5
6	211.8	295.3	6.3	0.5
7	213.4	296.1	9.5	0.5
8	211.8	295.3	12.6	0.5
9	210.3	295.2	9.5	0.5
10	213.4	294.8	6.3	0.5
11	216.4	294.3	6.3	0.5

M. Tabesh / A. Dolatkhahi

Table 2. Pipe data

Pipe No.	Diameter (mm)	Length (m)	CHW
1	457.2	61	100
2 (pump)	457.2	1	100
3	457.2	3209.5	100
4	355.6	1609.3	100
5	254	1609.3	100
6	254	1609.3	100
7	304.8	1609.3	100
8	203.2	1609.3	100
9	254	1609.3	100
10	304.8	1609.3	100
11	203.2	1609.3	100
12	152.4	1609.3	100
13	152.4	1609.3	100

To illustrate the differences between DDSM and HDSM based models, the results of hydraulic analysis for nodes 4 and 11, and water level at tank 1 are shown in Figures 4 and 5 for a period of 24 hours. It can be seen in Figure 4 that for normal conditions the results of HDSM and DDSM are more or less the same. However, when considering leakage in the hydraulic analysis, the pressure drops dramatically because of excess head loss. As a result, velocity values in pipes would be higher in HDSM with leakage analysis. Table 3 illustrates the discharge and velocity values of each pipe for daily average demands from different hydraulic analyses.

According to the variations of the water level in tank 1 and the upward trend during the first 12 hours, it can be seen that in both HDSM and DDSM the network is fed by the pump and reservoir. The increase in nodal heads confirms this fact. At 12 a.m. the pump is turned off and the network is fed by tank 1. This is because the water level is at the maximum level of 42.672 m. This situation continues until 11:00 p.m. when the tank water level is equal to the minimum level of 33.528 m. In this period all nodes face a decrease in pressure. At this time, the pump is turned on again and the network is fed by a reservoir which leads to an increase in nodal heads.

When leakage is incorporated in the HDSM procedure, the network faces an increase in pressure and available discharge. As a result, the reservoir can feed the network till 2:00 p.m. and tank 1 will be operational from 2 to 12 p.m.
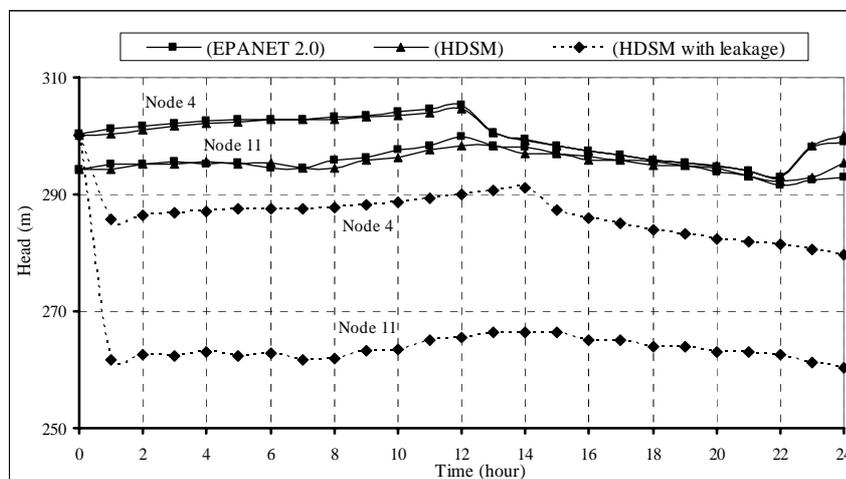

Fig. 4. Comparison of hydraulic analysis results at nodes 4 and 11

Table 3. Comparison of different hydraulic analysis results for pipes

Pipe No.	Discharge (l/s)			Velocity (m/s)		
	EPANET 2.0	HDSM	HDSM with leakage	EPANET 2.0	HDSM	HDSM with leakage
1	48.35	48.10	50.80	0.29	0.29	0.32
3	117.74	116.53	129.08	0.72	0.71	0.78
4	77.87	77.20	79.08	0.78	0.77	0.82
5	8.16	8.13	9.01	0.16	0.16	0.20
6	30.41	30.36	31.22	0.60	0.60	0.63
7	11.90	11.78	12.80	0.16	0.16	0.20
8	1.85	1.74	2.95	0.06	0.06	0.10
9	12.06	11.98	12.58	0.24	0.24	0.26
10	7.61	7.56	8.66	0.10	0.10	0.14
11	8.88	8.76	9.82	0.27	0.27	0.30
12	3.73	3.44	9.15	0.20	0.18	0.23
13	2.57	2.27	5.75	0.14	0.12	0.18

A practical approach is adopted to run simulations over a 72 hour period and then only the final 24 hours is used as being representative of the system in a stable (equilibrium) state for water quality parameters such as water age and chlorine concentration. The values of residual chlorine at nodes 4 and 11 are presented in Figure 6. It can be observed that the residual chlorine value from HDSM with leakage is less than DDSM and HDSM at most periods of time. The sudden decrease is because of feeding the system by tank. This situation has occurred sooner at node 4 because it is closer to the tank.

To determine the water age, the initial concentration values at all nodes and tanks are ideally considered as zero at the beginning of the analysis. The results of water age for nodes 4 and 11 during the third 24 hour period from three different simulations can be seen in Figure 7. Low water age for the first part of the day is because of the feeding of the network from the reservoir. Obviously feeding by the tank leads to a sudden increase of water age later on in the day.
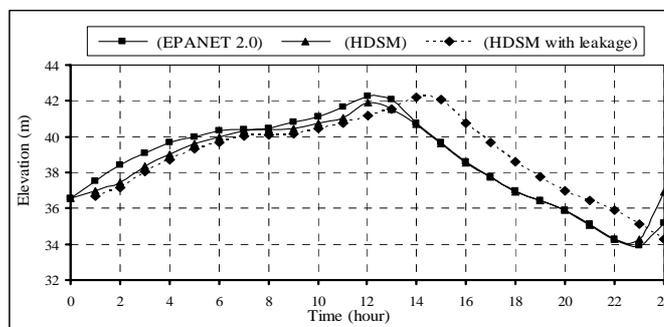

Fig. 5. Comparison of hydraulic analysis results for water elevation in tank 1
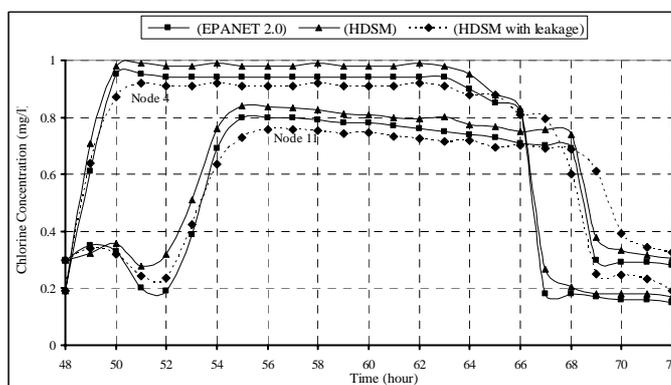

Fig. 6. Comparison of analysis results for chlorine concentration values at nodes 4 and 11

The performance indices for residual chlorine and water travel time are presented in Figs. 8 and 9. In these figures, the differences between HDSM and DDSM based quality models are illustrated. Figure 8 shows the network performance index for residual chlorine values. The unacceptable performance between 50-66 hours is because of an excess of residual chlorine from the standard upper level (0.5 mg/l) (Fig. 6). It can be seen that the network has a better performance with the DDSM model because of its lower values of residual chlorine in comparison with the HDSM. This situation is as expected because of high initial concentration values, i.e. 0.5 mg/l at nodes and 1 mg/l at the reservoir and tank, which are above the optimal range. It can be said that if the initial chlorine injected into the system is decreased, leading to a decrease of nodal residual chlorine from the standard values (0.2-0.5 mg/l), the performance index and the conclusion would be different. Also, performance improvement for models of HDSM with leakage will happen because of a decrease in residual chlorine values.
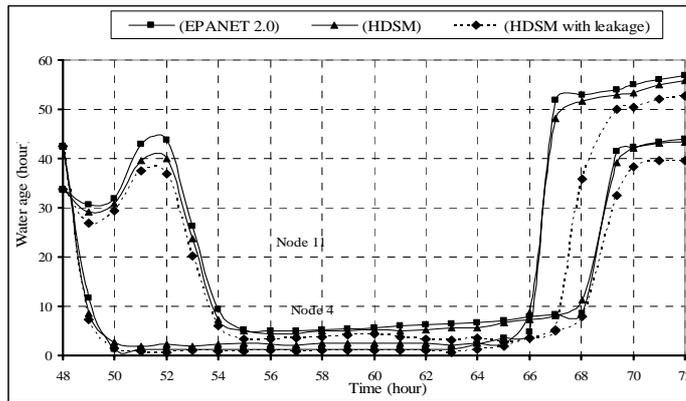

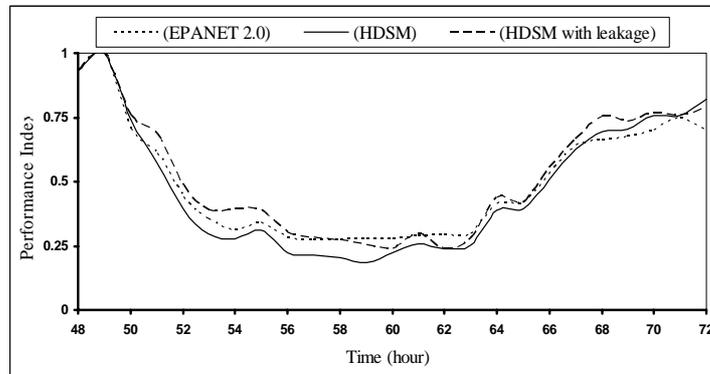Fig. 7. Comparison of analysis results for water age at nodes 4 and 11

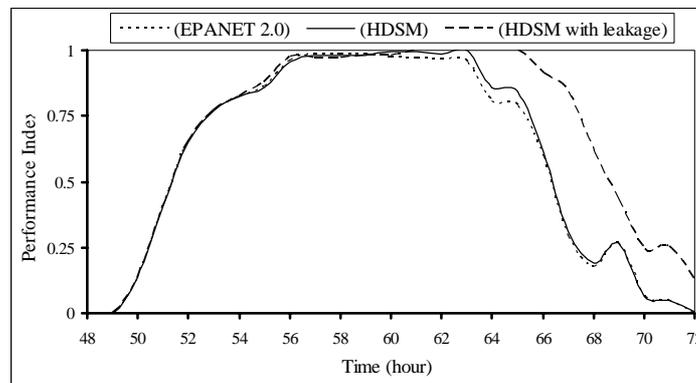Fig. 8. Network performance index based on residual chlorine

Fig. 9. Network performance index based on water age

Figure 9 illustrates the performance index based on water age. It can be seen that in the first 8 hours of the day, when the pump is on and the reservoir feeds the network, water age decreases and the performance index increases from a totally unacceptable level to the highest level. However, after that time the performance index decreases by the end of the day when a totally unacceptable level arises because the pump is off and tank 1 feeds the network. To improve this situation, an extra tank can be used or the pump may be operated for the entire 24 hour period. In the case of HDSM with leakage, the performance index is higher because of the higher velocity values, in comparison with the DDSM and HDSM models [21, 22].

4. SUMMARY AND CONCLUSIONS

In this paper, the formulation of quality analysis of water distribution networks was incorporated into the HDSM hydraulic model. This conjunctive model is also able to evaluate leakage in the network. According to the results of the case study, it can be said that in normal situations there are no meaningful differences between the DDSM and HDSM results. When a leak is considered, the pipe velocity is higher than the DDSM results. Therefore, water age is decreased and the water age performance index is improved. For a realistic assessment of network performance, it is proposed that a combination of quality parameters should be considered simultaneously, and any conclusion based on just one parameter (especially water age) may be misleading.

It can be concluded that in abnormal conditions the residual chlorine from the HDSM is less than the DDSM. In the case of excess in demand, simulations based on DDSM and HDSM with leakage show higher velocity in pipes, which lead to lower water age. In this situation the chlorine performance is improved if the initial values of chlorine concentration are above the standard rate. Otherwise, with higher differences from standard values, the performance index will be decreased.

REFERENCES

1. Germanopoulos, G., Jowitt, P. W. & Lumbers, J. P. (1986). Assessing the reliability of supply and level of services for water distribution system. *Proceeding ICE, Part 1*, No. 80, 413-428.

2. Wagner, J. M., Shamir, U. & Marks, D. H. (1988). Water distribution reliability: Analytical methods. *Journal of Water Resources Planning and Management, ASCE, 114*(3), 235-275.

3. Cullinane, M. J., Lansey, K. E. & Mays, L. W. (1992). Optimization-availability-based design of water distribution networks. *Journal of Hydraulic Engineering, ASCE, 118*(3), 420-441.

4. Tanyimboh, T. T., Tabesh, M. & Burrows, R. (2001). An appraisal of the source head method for calculating the reliability of water distribution networks. *Journal of Water Resources Planning and Management, ASCE, 127*(4), 206-213.

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

6. Males, R. M., Clark, R. M., Wehrman, P. J. & Gates, W. E. (1985). Algorithm for mixing problems in water systems. *Journal of Hydraulic Division, ASCE, 111*(2), 206-219.

7. Shah, M. & Sinai, G. (1988). Steady state model for dilution in water networks. *Journal of Hydraulic Division, ASCE, 114*(2), 192-206.

8. Boulos, P. F., Altman, T. & Sadhal, K. (1992). Computer modeling of water quality in large multiple source networks. *Journal of Applied Mathematical Modeling, 16*(8), 439-445.

9. Rossman, L. A., Boulos, P. F. & Altman, T. (1993). Discrete volume element method for network water-quality

models. *Journal of Water Resources Planning and Management, ASCE, 119*(5), 505-517.

10. Liou, C. P. & Karoon, J. R. (1987). Modeling the propagation of waterborne substances in distribution networks. *Journal of American Water Works Association, 79*(11), 54-58.

11. Grayman, W. M., Clark, R. M. & Males, R. M. (1988). Modeling distribution system water quality: dynamic approach. *Journal of Water Resources Planning and Management, ASCE, 114*(3), 295-312.

12. Boulos, P. F., Altman, T., Jarrige, P. A. & Collevati, F.(1995). Discrete simulation approach for networks water quality models. *Journal of Water Resources Planning and Management, ASCE, 121*(1). 49-60.

13. Islam, M. R. & Chaudhry, M. H. (1998). Modeling of constituent transport in unsteady flows in pipe networks. *Journal of Hydraulic Engineering, ASCE, 124*(11), 1115-1124.

14. Rossman, L. A. & Boulos, P. F. (1996). Numerical methods for modeling water quality in distribution system. *Journal of Water Resources Planning and Management, ASCE, 122*(2), 137-146.

15. Tabesh, M. (1998). Implication of the pressure dependency of outflows on data management, mathematical modeling and reliability assessment of water distribution system. Ph.D. Thesis, Department of Civil Engineering, University of Liverpool, UK.

16. Tabesh, M., Tanyimboh, T. T. & Burrows, R. (2002). Head driven simulation of water supply networks, International *Journal of Engineering, Transaction A: Basics, 15*(1), 11-22.

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

18. Germanopoulos, G. (1985). A technical note on the inclusion of pressure dependent demand and leakage terms in water supply network models. *Civil Engineering Systems, 2*, Sep., 171-179.

19. Bhave, P. R. (1991). Analysis of flow in water distribution networks. Technomic Publishing Co., Lancaster, Pa.

20. Dolatkhahi, A. (2002). Assessment the effect of pressure dependent hydraulic models on water quality performance of water distribution networks. MSc. Thesis, Civil Engineering Department, University of Tehran, Iran, (in Persian).

21. Dolatkhahi, A. & Tabesh, M. (2004). Performance assessment of water distribution networks based on quality parameters using head driven simulation method. Proceedings of the World Water and Environmental Resources Congress, ASCE, Salt Lake City, Utah, June 27 - July 1.

22. Tabesh, M. & Dolatkhahi, A. (2003). Analysis of quality parameters of water distribution networks using a pressure dependent hydraulic model. *Proceedings of the 6th International Conference on Civil Engineering*, D. Mostofinejad (ed.), Isfahan University of Technology, Isfahan, Iran, 5-7 May, 315-322, (in Persian).