

## **Impact of Water Demand Pattern Variation on Hydraulic Behavior and Water Quality in Water Distribution Systems**

**Hassan I. Mohamed<sup>1</sup>, Mohamed H. Abdel-Aal<sup>1</sup>, and Amr M. El-Darder<sup>2\*</sup>**

<sup>1</sup> Civil Eng. Department, Assiut University, Egypt

<sup>2</sup> Holding Company for Drinking Water and Sanitary Drainage, Assiut and New Valley Branch

**Received:** 29/12/2020

**Accepted:** 2/2/2021

© Unit of Environmental Studies and Development, Aswan University

### **Abstract:**

Water is the most valuable source for human life. Egypt is one of the driest countries facing an enormous shortage of potable water especially with the population increase and limited water resources. It is becoming aware of the need to preserve water and sustainable water management. Potable water supply pipes are important for delivering water to consumers and serving as the last process between the treatment plant and the consumer. Water demand can vary periodically over the course of the day and for the same average daily demand, the peak value of the demand pattern can vary. These variations in peak demand can have significant effect on the hydraulic behavior of pipes network, water quality, and energy consumption through it. A significant amount of cost is spent on energy consumption in the water supply pipes network. In this research, the effect of water demand pattern variation on hydraulic behavior, water quality and energy consumption in water supply pipes network is studied. EPANET2 software was used to perform the hydraulic analysis through extended period simulation through a hypothetical water supply pipes network. Five patterns of demand variation were adopted in this analysis called pattern demand for big city, village, rural area, industrial zone and hotels. It was found that the type of demand pattern has a significant effect on velocity through the different pipes and pressure head of the different nodes and chlorine decay through the network.

**Keywords:** pips network, water demand, quality

### **1- INTRODUCTION**

There are many potential concerns with the operation of the water supply pipes network. One of these concerns is hydraulic in nature i.e. pressure and flow, the other is energy consumption in the network, and that of a water quality nature i.e. concentration magnitudes and time of arrival at various locations. These concerns, however, are directly affected by the water demand pattern. It is, therefore, imperative that the effect of demand patterns variation on hydraulic behavior and energy consumption be quantified.

**Corresponding author\*:** E-mail address: [amraldardery@yahoo.com](mailto:amraldardery@yahoo.com)

Many researchers studied water supply pipes network from different view point, among them Molino et al. (1991), Colombo and Karney (2002), Ghimire and Barkdoll (2007), Martinez-Solano et al. (2008), Mohamed and Abozeid (2011), Ali et al. (2010), Gad and Mohamed (2014), Speight (2014) and Mohamed et al. (2019).

Roshani and Filion (2014) studied the effect of reduction in demand on the optimum water main asset rehabilitation decisions and costs. The relationship between reduction of demand and energy use, capital and operational costs in optimized system designs were examined. Many water demand reduction scenarios were simulated and the results indicated that water cost was not affected by a reduction in demand. Also, the annual capital cost and annul overall cost did not significantly change until demand was reduced by 25%.

Water demand management alleviates the current contradiction between water supply and demand effectively, and provides a good reference to address water use with low efficiency and the water crisis (Da-Ping et al., 2011). Figure (1) shows the implementation mechanism of water demand management. This mechanism can be implemented in domestic water sector through water rationing planning, seasonal price and price ladder according to the characteristics of the city formulating policies. This encourages households to use water-saving devices to replace traditional ones. Accelerate the technological transformation of urban water supply networks; reduces leakage in water transmission and distribution, and increase the urban sewage treatment and recycling efforts.

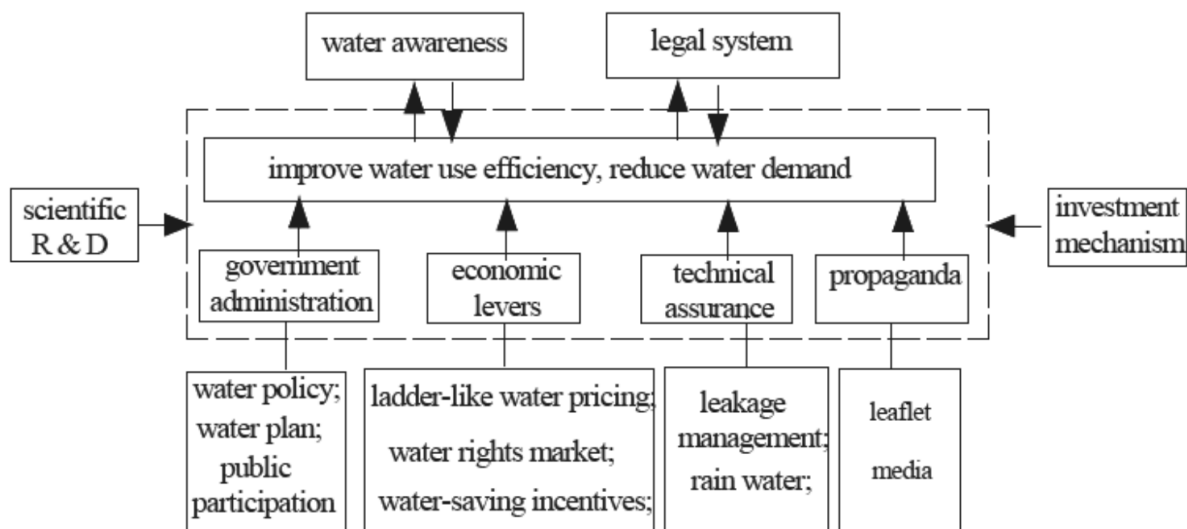


Fig. (1): Implementation mechanism of water demand management [Da-ping et al. 2011].

Sapkota et al. (2015) reviewed hybrid water supply system and its impacts in meeting water demand increase. It was found that hybrid water supply systems can have implications on the operation and performance of the existing centralized infrastructure. They conclude that the interactions between decentralized and centralized systems are highly complex. Furthermore, implementing hybrid water supply systems has other challenges including energy usage, operational performance, asset management, cost, and public acceptance. To overcome these challenges, a better understanding of the overall system is inevitable, which highlights the need for valid assessment criteria of hybrid water supply systems in terms of the interaction between the decentralized and centralized systems. El-Nwsany et al. (2019) putted guidelines to facilitate the management of sustainable water in schools. A guidance on the application of sustainable water utilization and sustainable drainage, during the stages of school design and operation was

included. They recommended that the use of gray water and waste water ought to be taken into consideration because the water crisis, which can increase within the upcoming years and every drop of water, will be valuable.

Kumarasamy et al. (2017) examined conservation methods as ways of optimizing access to clean water. The main water conservation factors focused on using water efficient technologies and alternative forms of collection, recycling and storage of water for later use. Investigation of each system individually and together can provide insight on percentages of potable water that could be saved.

## 2- THEORETICAL APPROACH

Hydraulic models are important for studying of water pipe networks. This is due to the complex topology, frequent changes and size of water distribution networks (Harding and Walski, 2000). Hydraulic models are utilized for various important tasks by engineers, such as the design of new and analysis of existing distribution networks, long term master planning and operational planning.

### 2.1- Hydraulic Model

Many computer softwares have been developed to simulate pipes network. The used models can be divided into four basic types:

- Steady-state models
- Extended-period simulation models
- Water quality models
- Optimisation models

One of the most widely used models is the EPANET model which can simulate steady state conditions, extended period, and water quality (Rossman et al., 1993; Rossman, 1994; and Rossman, 2000). EPANET is developed by the U.S. Environmental Protection Agency. The hydraulic bases of this software are explained in the following. Pipe network is assumed have  $N$  junction nodes and  $NF$  fixed grade nodes (tanks and reservoirs). The flow-head loss relation in a pipe between nodes  $i$  and  $j$  can be given as:

$$H_i - H_j = h_{ij} = aQ_{ij}^n + mQ_{ij}^2 \quad (1a)$$

where  $H$  = nodal head,  $h$  = headloss,  $a$  = resistance coefficient,  $Q$  = flow rate,  $n$  = flow exponent, and  $m$  = minor loss coefficient. For pumps, the head loss (negative of the head gain) is represented by a power law of the form

$$h_{ij} = -\omega^2 (h_o - b(Q_{ij} / \omega)^p) \quad (2b)$$

where  $h_o$  is the shutoff head for the pump,  $\omega$  is a relative speed setting, and  $b$  and  $p$  are the pump curve coefficients. The second group of equations that must be satisfied is the flow continuity at all nodes,  $N$ :

$$\sum_j Q_{ij} - D_i = 0 \quad \text{for } i=1, \dots, N \quad (3)$$

where  $D_i$  is the flow demand at node  $i$  and by convection, flow into a node is positive. The method used in EPANET to simultaneously solve the flow continuity and head loss equations, Eq.(1) and Eq.(3), that characterize the hydraulic state of the pipe network at a given position in time is called "gradient method" (Salgado et al. 1988). For time variation simulation, EPANET hydraulic model is used to compute junction heads and pipe flows for a fixed set of reservoir

levels, tank levels, and water demands over a succession of points in time. From time step to the next, water levels in reservoirs and demands at nodes are updated according to their prescribed time patterns while tank levels are updated using the current flow solutions.

## 2.2- Water Quality Model

*EPANET* water quality module uses a Lagrangian approach to follow the chemical concentration in discrete parcels of water as they flow along pipes and mix together at nodes between fixed-length time steps. The concentration at each element is subjected to reactions, the total content of mass and flow volume entering and leaving the node is kept constant, and the segments position is updated, for different time steps. The chlorine concentration leaving the node is the flow-weighted sum of the concentrations from inflow into pipes and external sources. For a specific node  $k$  one can write:

$$C_{i|x=0} = \frac{\sum_{j \in I_k} Q_j C_{j|x=L_j} + Q_{k, ext} C_{k, ext}}{\sum_{j \in I_k} Q_j + Q_{k, ext}} \quad (4a)$$

where  $i$  = link with flow leaving node  $k$ ,  $I_k$  = set of links with flow into  $k$ ,  $L_j$  = length on link  $j$ ,  $Q_j$  = flow in link  $j$ ,  $Q_{k, ext}$  = external source flow entering the network at node  $k$ , and  $C_{k, ext}$  = concentration of the external flow entering at node  $k$ . The notation  $C_{i|x=0}$  represents the concentration at the start of link  $i$ , while  $C_{i|x=L_i}$  is the concentration at the end of the link.

The concentration throughout a storage tank, under completely mixed conditions is a blend of the current contents and that of any entering water. At the same time, the internal concentration could be changed due to reactions,  $r(C_s)$ . Mass conservation at the storage tanks is given by:

$$\frac{\partial(V_s C_s)}{\partial t} = \sum_{i \in I_s} Q_i C_{i|x=L_i} - \sum_{j \in O_s} Q_j C_s + r(C_s) \quad (4b)$$

where  $V_s$  = volume in storage at time  $t$ ,  $C_s$  = concentration within the storage facility,  $I_s$  = set of links providing flow into the facility, and  $O_s$  = set of links withdrawing flow from the facility.

When *EPANET* quality model is applied to a network as a whole, Eq.(4) and Eq.(5) are solved for  $C_i$  in each pipe  $i$  and  $C_s$  in each storage facility  $s$ . The solution is subjected to the initial conditions that specify  $C_i$  for all  $x$  in each pipe  $i$  and  $C_s$  in each storage facility  $s$  at time 0. Also, the solution is subjected to boundary conditions that specify values for  $C_{k, ext}$  and  $Q_{k, ext}$  for all time  $t$  at each node  $k$ , which has external mass inputs. The hydraulic simulation model provides the volume  $V_s$  in each storage facility  $s$  and the flow  $Q_i$  in each link  $i$  at all time  $t$ . From one-step to the next the concentration in each element, node, and tank are updated.

## 2.3 Decay Reactions of Residual Chlorine

The loss of *RCC* along drinking water distribution networks (*DWDNs*) is processed in two mechanisms as; chlorine reactions in bulk fluid and chlorine reactions with pipe walls. Transport of chlorine along the  $i$ th pipe is given by the classical advection equation (Rossman et al. 1993, and Rossman and Boulos 1996):

$$\frac{\partial C_i}{\partial t} = -u_i \frac{\partial C_i}{\partial x} + r(C_i) \quad (5)$$

where  $C_i = RCC$  in pipe  $i$  as a function of distance  $x$  and time  $t$ ,  $u_i =$  mean flow velocity in pipe  $i$ , and  $r =$  rate of residual chlorine reaction with both bulk fluid ( $r_b$ ) and pipe walls ( $r_w$ ) as a function of its concentration,  $C_i$ . The rate of residual chlorine bulk reaction,  $r_b$ , is commonly simulated by the following first-order decay equation (Castro and Neves 2003):

$$r_b = -k_b C_i \quad (6)$$

where  $k_b$  is a bulk reaction rate coefficient. For first-order kinetics (Rossman et al. 1994). The rate of residual chlorine wall reaction,  $r_w$ , can be expressed as:

$$r_w = \frac{-4k_w k_f C_i}{d(k_w + k_f)} \quad (7)$$

where  $k_w =$  wall reaction rate coefficient,  $k_f =$  mass transfer coefficient, and  $d =$  pipe diameter. While  $k_b$  can be determined from laboratory measurements,  $k_w$  is based on local residual chlorine measurements. Mass transfer coefficient,  $k_f$ , can be expressed in terms of a dimensionless Sherwood number,  $Sh$ , as:

$$k_f = Sh \frac{D}{d} \quad (8)$$

in which  $D$  is the diffusivity of chlorine. In fully laminar flow, the average Sherwood number along the length of a pipe ( $L$ ) can be expressed as:

$$Sh = 3.65 + \frac{0.0668(d/L)ReSc}{1 + 0.04[(d/L)ReSc]^{2/3}} \quad (9)$$

in which  $Re =$  Reynolds number and  $Sc =$  Schmidt number (kinematic viscosity of water divided by the diffusivity of chlorine). For turbulent flow, the empirical correlation of Notter and Sleicher (1971) is used:

$$Sh = 0.0149 Re^{0.88} Sc^{1/3} \quad (10)$$

### 3- CASE STUDY

#### 3.1 Description of The Used Water Distribution Network

This research is based on the distribution network shown in Fig. (2). There is one source of water feeding the network, from which water is pumped into the network, node 1. The ID labels for the various components are shown in the figure. Elevations of all the network junctions are assumed to be the same at level zero. Average base demands for the different junction nodes are shown A to I are 0.0, 49.21, 21.65, 5.22, 38.31, 12.96, 63.8, 13.13, 41.25, and 18.66 liters/sec., respectively.

The distribution system is shown in Fig. (2), is composed of different diameter pipelines. All pipes are UPVC with lengths of P2 through P16 as shown in table (1). The head loss in each pipe is computed using Darcy-Weisbach formula with roughness height values of 0.03 for all pipes. Diameters of pipes P2 to P16 are shown in table (1). There one source of water reservoir 1 shown in Fig. (2). In addition, one pumping station having a characteristic curve as shown in table (2).

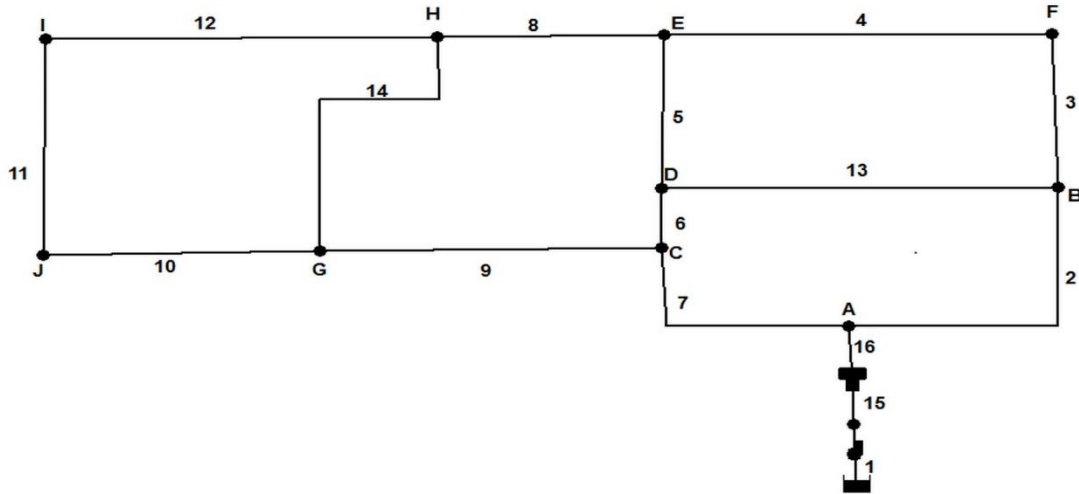


Fig. (2): Schematic diagram for the used pipes network.

Table (1): Lengths and diameters of the different pipes in the network.

pipe ID	Length(m)	Diameter(mm)	$K_s$
P2	966.25	560	0.03
P3	600	500	0.03
P4	540	500	0.03
P5	575	250	0.03
P6	107.5	355	0.03
P7	910	560	0.03
P8	530	450	0.03
P9	747.5	355	0.03
P10	743.75	250	0.03
P11	710	125	0.03
P12	935	280	0.03
P13	577.5	250	0.03
P14	907.5	280	0.03
P15	50	580	0.03
P16	50	580	0.03

Table (2): Characteristic of the used pump

Pump	
Q (l/sec.)	H (m)
0	62
264	43
320	36

### 3.2 Diurnal Curve of Demands

A diurnal curve that makes demands at the junction nodes of the used network vary in a periodic way over the course of the day has to be used to make the network more realistic for analyzing an extended period of operation. Five scenarios for the demand curve with different peak values and hour of peak values as shown in Fig. (3) are used for simulation in this study. The curves provide demand peaking factors as the multipliers that are applied to the average base demand given for

each junction node in the distribution network to determine its actual demand in a given time period.

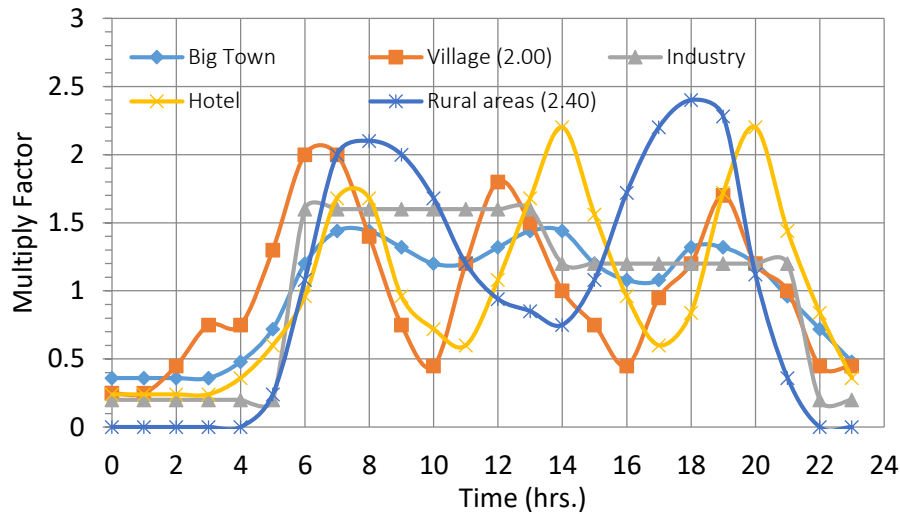


Fig. (3): Diurnal demand curve for extended period simulation for different Scenarios.

### 3.3 Residual Chlorine Reaction Coefficients

Chlorine decay simulation conducted with *EPANET*'s quality model takes into consideration the phenomena of chlorine reaction with chemical species in bulk fluid and pipe walls. Both  $k_b$  and  $k_w$  can depend on water characteristics, temperature, and the latter can be correlated to the pipe age and material. Reported reaction rate coefficients,  $k_b$  and  $k_w$ , for first-order decay of residual chlorine are presented in Table 3. Based on these reported values,  $k_b$  and  $k_w$  in this study are taken as  $-0.1008$  /day and  $-0.20$  m /day, respectively.

Table 3. Summary of the reported values of chlorine reaction rate coefficients.

$k_b$ ( 1/day)	$k_w$ (m/day)	Reference
-0.3432	-184.34	Castro and Neves (2003)
-0.40	-2.50	Basiouny and El-Atreby (2001)
-0.60	-0.2 ~ -2.0	Basiouny and El-Atreby (2000)
-0.1008	Na	Frederick et al. (1992)
-0.55	-0.15	Ozdemir (2005)

Na: not available.

It is assumed that the hydraulic analysis and chlorine concentration solution of the supply network may be carried out within a 1 hr. report time interval. However, the quality (concentration) calculations were performed at 5 min. intervals. The network hydraulics and concentration changes were assumed to be continuous for 72 hr. and only the last 24 hr. were considered. The reason for this is the long residence times in some pipes.

## 4- RESULTS AND DISCUSSIONS

### 4.1 Effect of Demand Patterns on Velocity in Pipes Network

Figures (4) to (8) show the effect of demand pattern variation on flow velocity in the pipes network at 8:00 AM for example. Figure (4) shows the flow velocity at different pipes for the



case of big town demand pattern, Figure (5) is for hotels demand, Fig. (6) is for industrial zone demand, Fig. (7) is for rural area demand and Fig. (8) is for village demand pattern. It can be shown from Fig. (4) that all of the pipes have a velocity less than 1.0 m/s except pipe number 9 has velocity larger than 1.0 m/s and three pipes have velocity less than 0.5 m/s and the same can be noticed for the case of hotel and industrial demand in Fig. (5) and (6). However, for rural area demand (Fig. 6), the number of pipes with velocity higher than 1.0 m/s increased. For village demand (Fig. 8), all the pipes have velocity less than 1.0 m/s.

Figs. (9) and (10) show the time variation of velocity in pipe 9 and 11, respectively, for different demand patterns shown in Fig. (3). It can be noticed from Fig. (9) that the velocity in pipe follows the same trend of demand pattern and the velocity in this pipe is higher than 0.5 m/s for most of the day. In contrary to this, as shown in Fig. (10) the velocity in pipe 11 at the end of the pipes network is nearly constant all the day with an average value of 0.35 m/s except for hotel demand pattern.

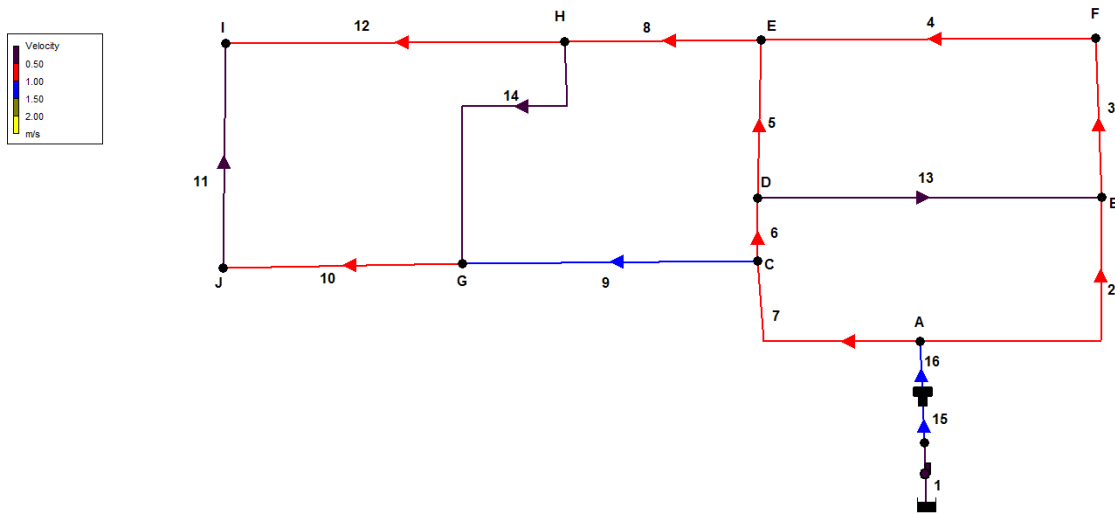


Fig. (4): Velocity in different pipes at hour 8:00 AM for big town demand pattern.

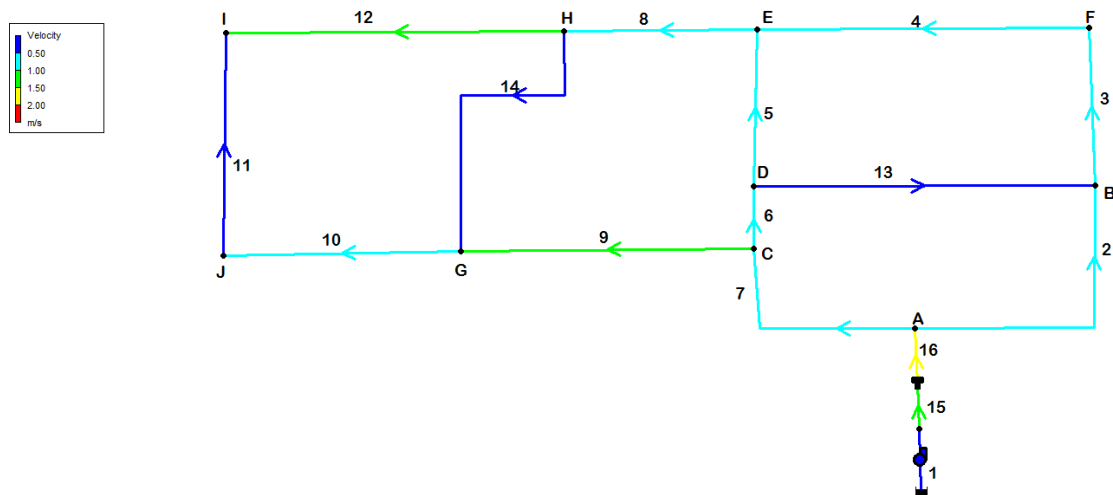


Fig. (5): Velocity in different pipes at hour 8:00 AM for hotels demand pattern.



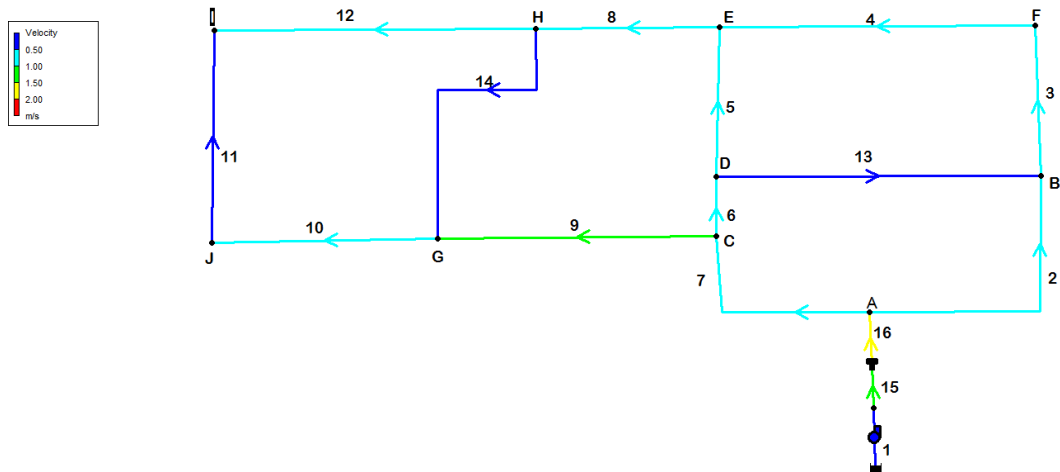


Fig. (6): Velocity in different pipes at hour 8:00 AM for industrial zone demand pattern.

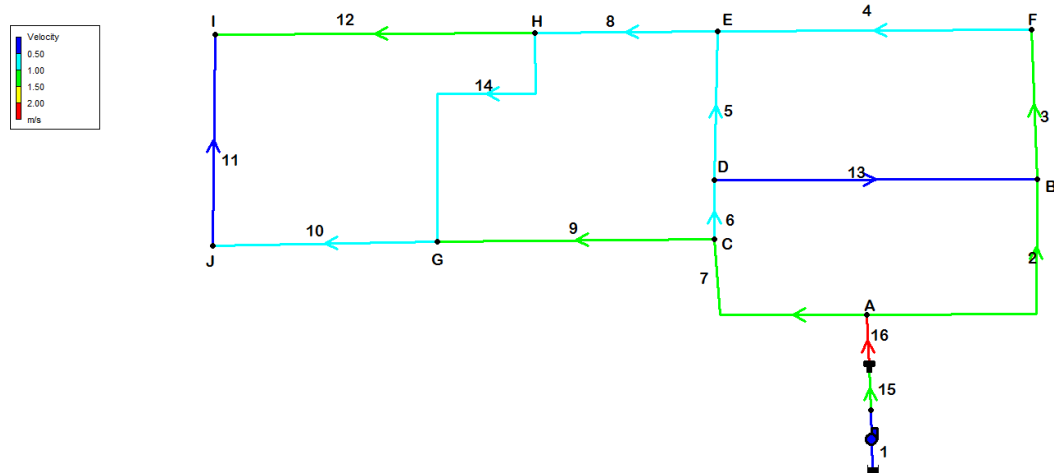


Fig. (7): Velocity in different pipes at hour 8:00 AM for rural area demand pattern.

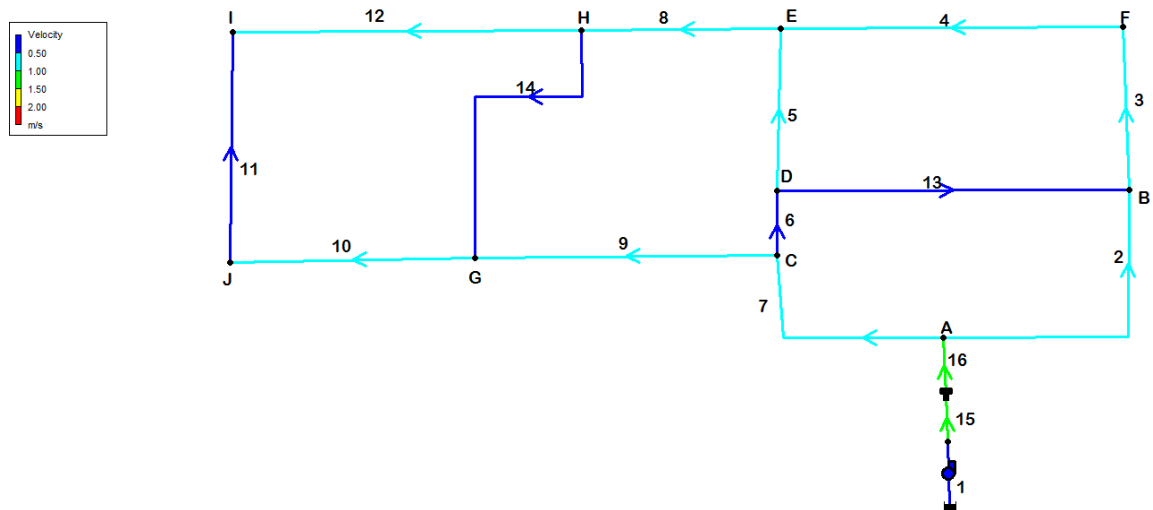


Fig. (8): Velocity in different pipes at hour 8:00 AM for village demand pattern.

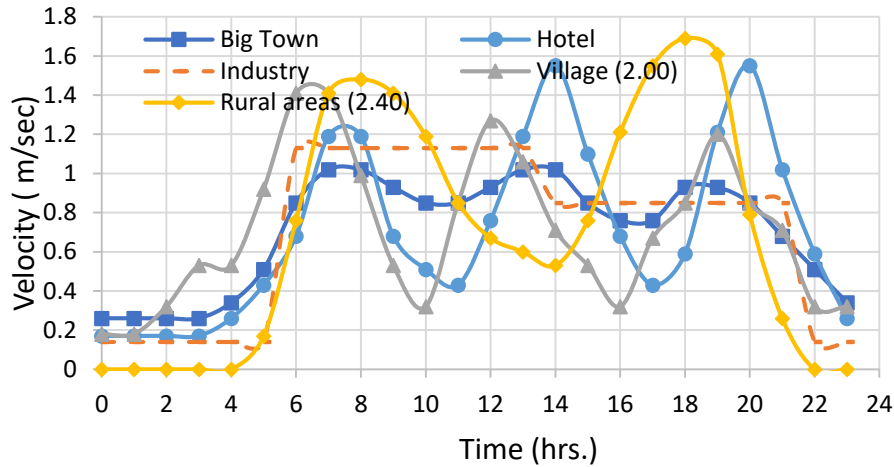


Fig. (9): Time variation of velocity in pipe 9 for different demand patterns.

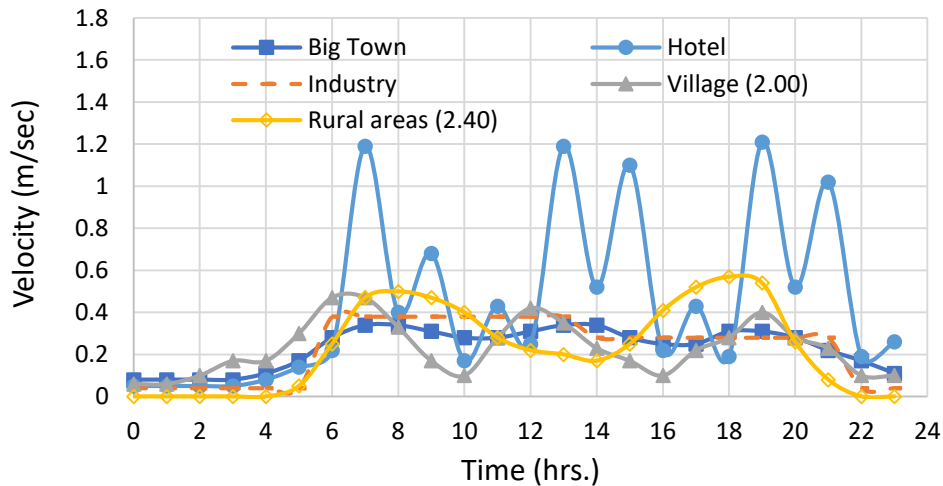
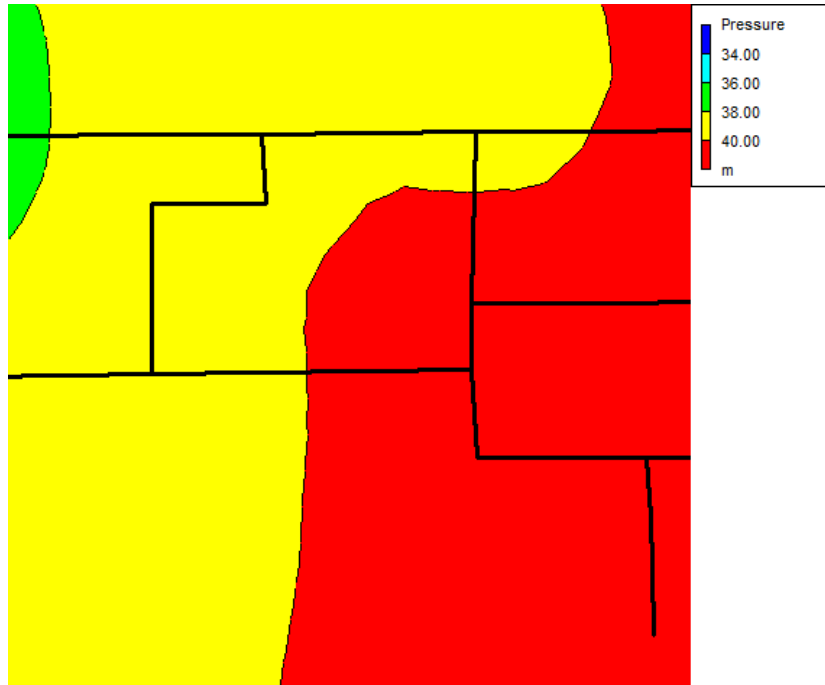


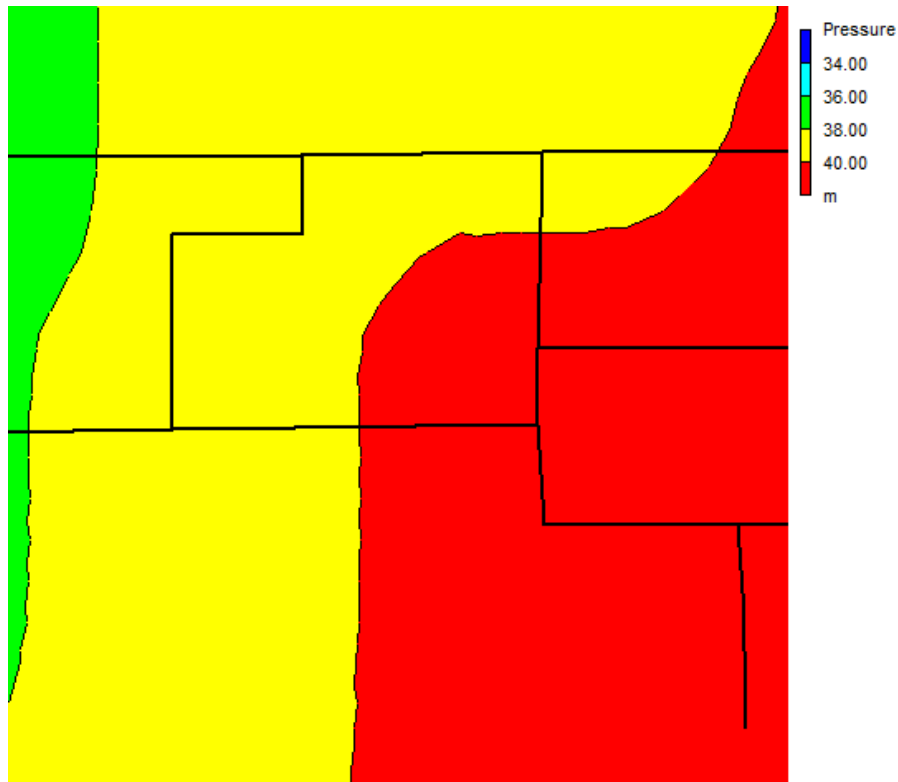
Fig. (10): Time variation of velocity in pipe 11 for different demand patterns.

#### 4.2 Effect of Demand Patterns on Pressure Head in Pipes Network

Figures (11) to (15) show the effect of demand pattern variation on pressure heads in the pipes network at 8:00 AM for example. Figure (11) shows the contour map of pressure head for big town demand pattern, Figure (12) is for hotels demand pattern, Fig. (13) for industrial zone, Fig. (14) for rural area and Figure (15) for village demand pattern. It can be shown from these Figs. That the pressure head higher than 36 m expect for rural demand is reduced to 34 m. The maximum pressure head is higher than 40 m for all demand patterns and the distribution of pressure head varies with demand pattern.



**Fig. (11):** Contour map of pressure head at hour 8:00 AM for big town demand pattern.



**Fig. (12):** Contour map of pressure head at hour 8:00 AM for hotel demand pattern.

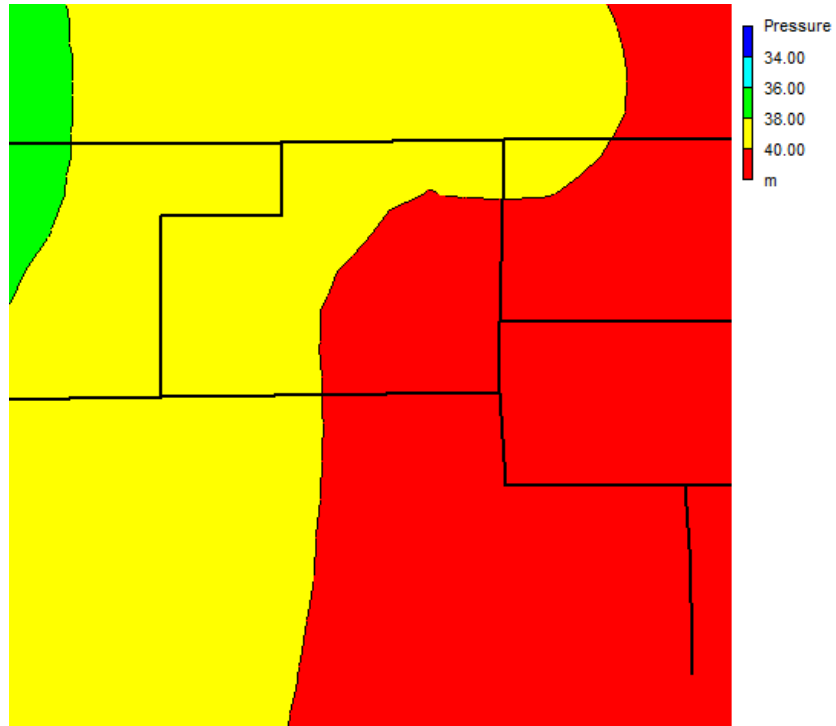


Fig. (13): Contour map of pressure head at hour 8:00 AM for industrial zone demand pattern.

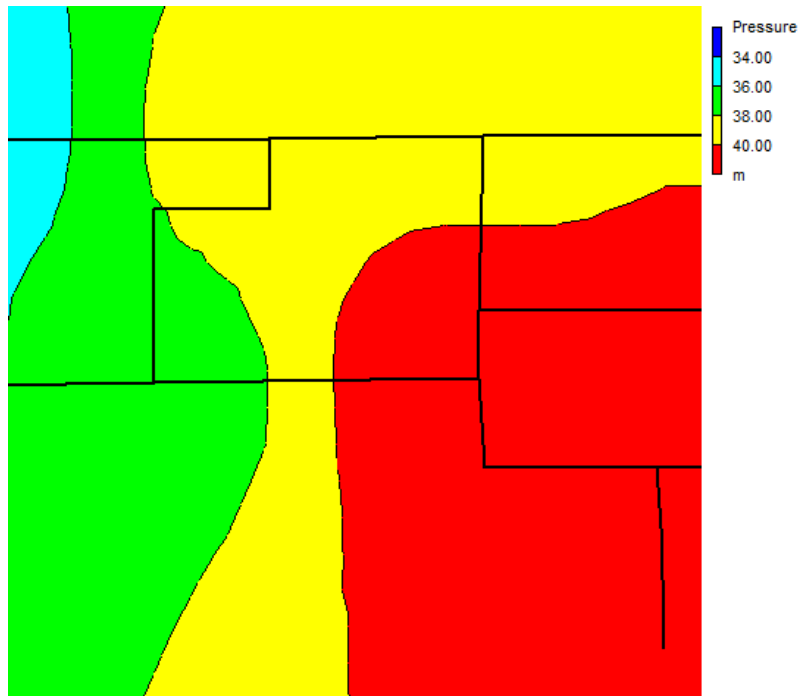


Fig. (14): Contour map of pressure head at hour 8:00 AM for rural area demand pattern.

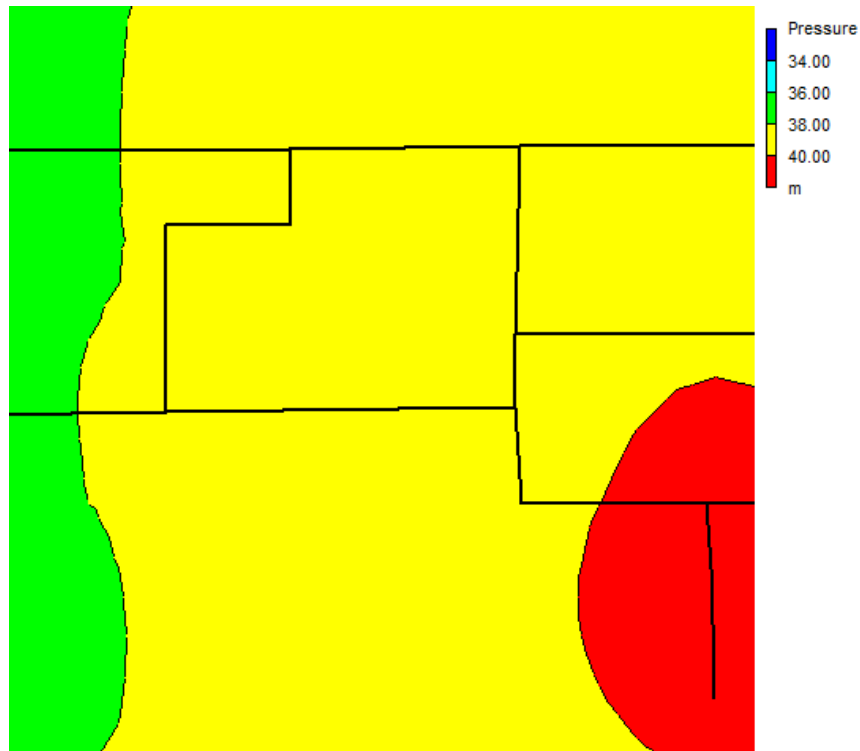


Fig. (15): Contour map of pressure head at hour 8:00 AM for village demand pattern.

Figs. (16) and (17) show the time variation of pressure head at nodes A and I, respectively, for different demand patterns shown in Fig. (3). It can be noticed from Fig. (16) that the pressure head follows the same trend of demand pattern and increase by increasing the peak demand and the variation is not so high as node A at the beginning of the network. However, at the end of pipes network as shown in Fig. (17), the variation in pressure head is so high through the day for all demand patterns and varied with demand pattern.

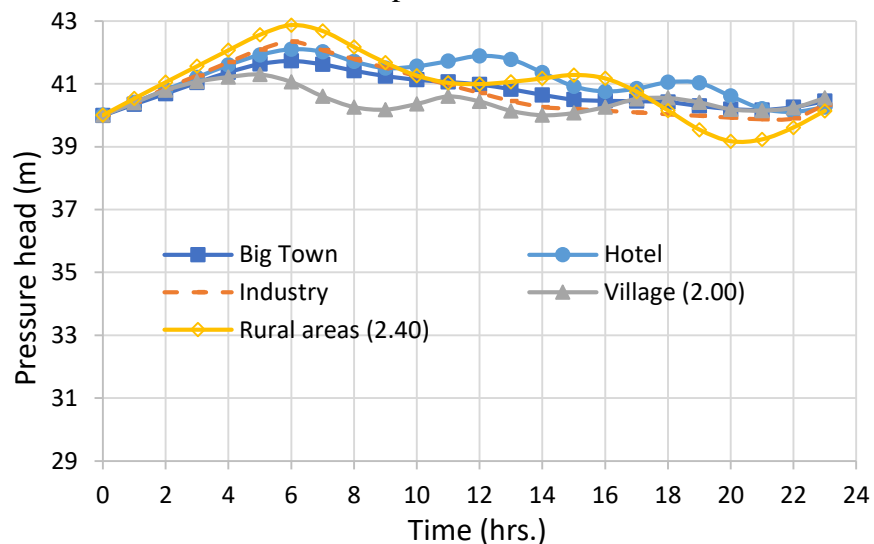


Fig. (16): Time variation of pressure head at node A for different demand patterns.

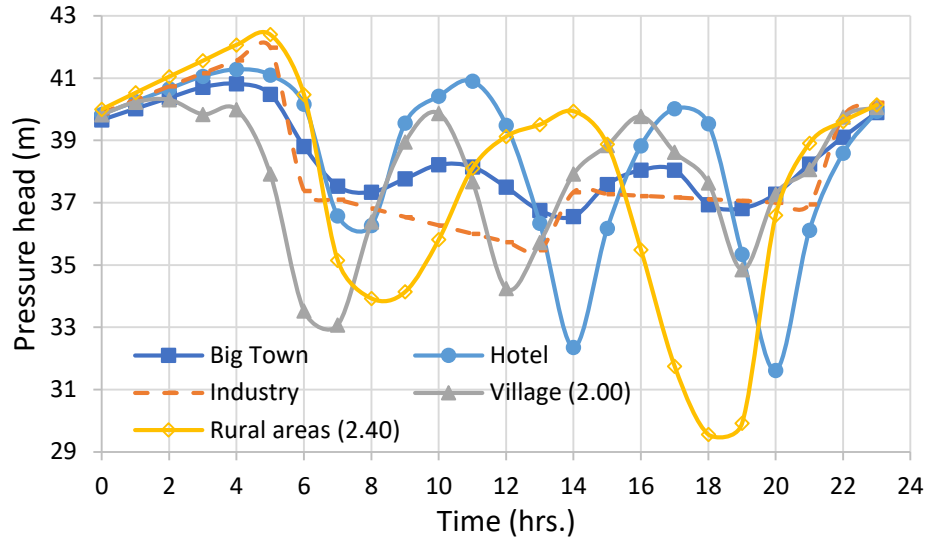


Fig. (17): Time variation of pressure head at node I for different demand patterns.

#### 4.3 Effect of Demands Patterns on Water Quality in Pipes Network

Besides effect of demand pattern variation on flow velocity in pipes network and pressure heads, it can affect the chlorine concentration in pipes network which is considered as an indicator for the water quality in the pipes network. Figs. (18) to (22) show contour maps for the chlorine concentration through the network at hour 8:00 AM for the five cases of demand patterns. It is noticeable from these figures that the chlorine concentration in the range of 0.4 to 0.44 mg/l for all demand patterns. However, the distribution is varied from case to case.

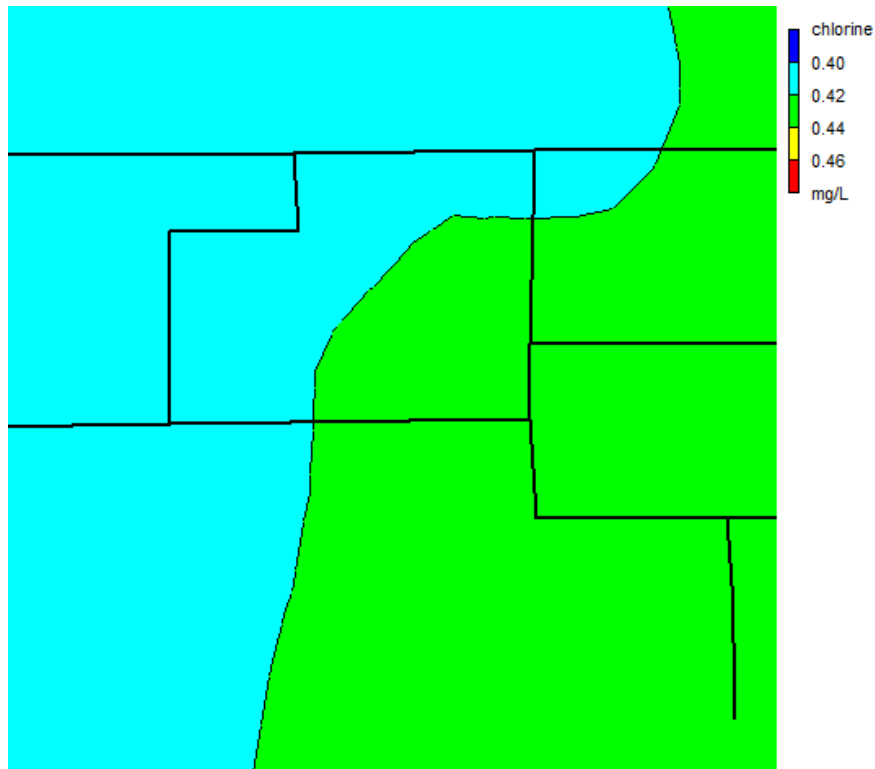


Fig. (18): Contour map of chlorine concentration at hour 8:00 AM for big town demand pattern.

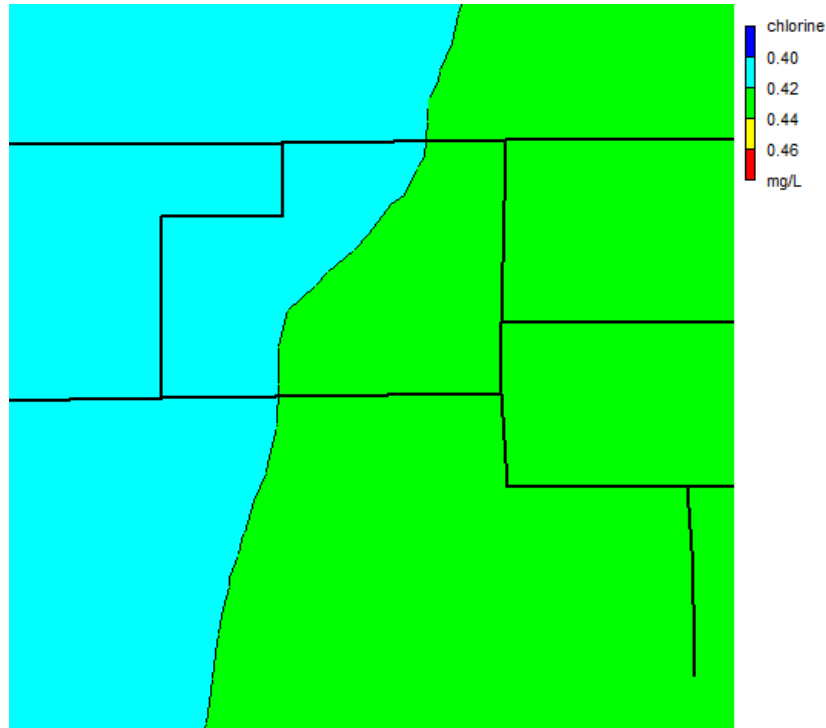


Fig. (19): Contour map of chlorine concentration at hour 8:00 AM for hotels demand pattern.

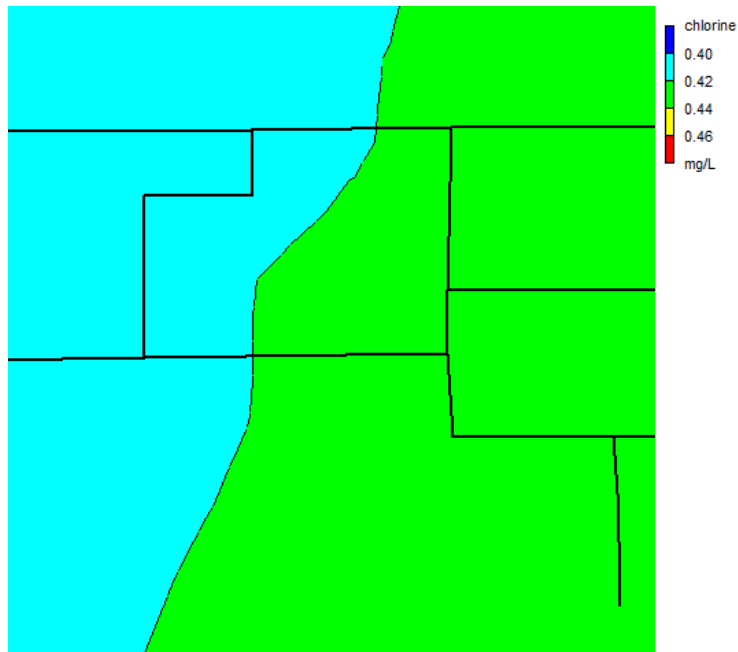


Fig. (20): Contour map of chlorine concentration at hour 8:00 AM for industrial zone demand pattern.



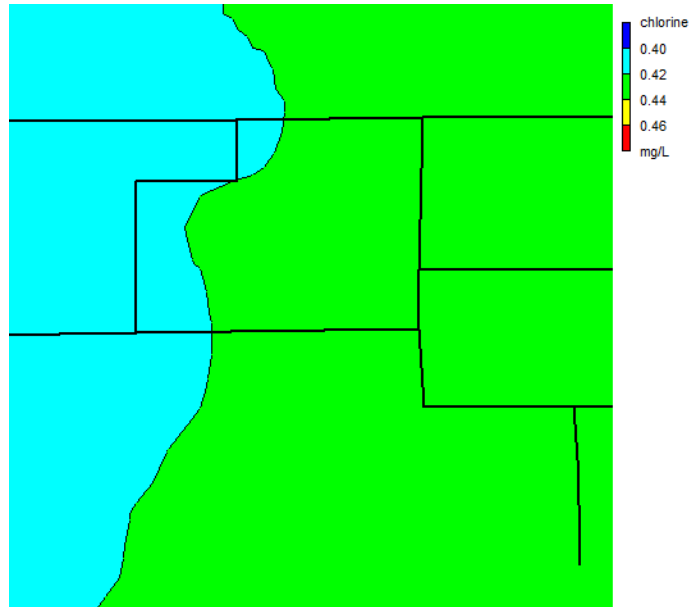


Fig. (21): Contour map of chlorine concentration at hour 8:00 AM for rural area demand pattern.

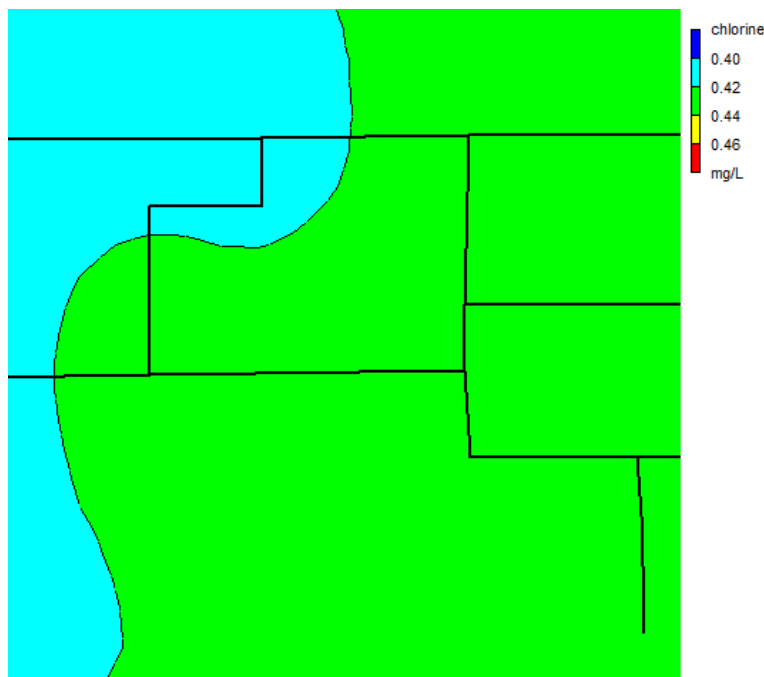


Fig. (22): Contour map of chlorine concentration at hour 8:00 AM for village demand pattern.

### Conclusions

Effect of water demand pattern variation on hydraulic behavior and water quality in pipes network is studied. Five types of demand patterns are considered, big town, hotels, industrial zone, rural area and village. For each pattern, the peak time and value is different. The main conclusions of this study can be summarized as follows;

1. The flow rate and mean velocity in pipes network are affected by variation in demand pattern peak cycle and time of peak.
2. The variation of velocity with time in pipes follow the same trend of demand pattern and this variation decreases as the flow rate decreased in pipes.

3. The pressure head is affected by demand pattern variation and the variation will be higher at the far end junctions compared with the beginning of the network.
4. The water quality in the pipes network can be affected by the variation in demand pattern.

## References

- Ali, N. A., Mohamed, H. I., El-Darder, M. E., and Mohamed, A. A. (2010). "Analysis of transient flow phenomenon in pressurized pipes system and method of protection", *Jour. of Eng. Science, Assiut University*, 38 (2), 323-342.
- Basiouny, M. and El-Etriby, H.K., 2000. "Modelling residual free chlorine in drinking water distribution systems". *Mansoura Engineering J., MEJ*, 25(1), pp. C1-C13.
- Basiouny, M.E.A. and El-Atreby, H.K., 2001. "Analysis of water distribution network performance by penalty curve". *Alexandria Engineering J.*, 40(2), pp. 201-214.
- Castro, P. and Neves, M., 2003. "Chlorine decay in water distribution systems case study–Lousada network". *Electron. J. Environ. Agric. Food Chem.* ISSN 1579-4377, 2(2).
- Colombo, A.F.; Karney, B. W. (2002). "Energy and costs of leaky pipes: Toward comprehensive picture", *Jour. of Water Resour. Plann.& Manag.*, 128 (6), ASCE, pp. 441-450.
- Da-Ping, X.; hong-yu, G. and Dan, H. (2011), "Discussion on the demand management of water resources", *Procedia Environmental Science*, 10, pp: 1173-1176.
- El-Nwsany, R. I., Maarouf, I., and Abd El-Aal, W., (2019), "Water management as a vital factor for sustainable school", *Alex. Eng. J.*, 58, pp: 303-3013.
- Frederick, L.H., Wayne, A.W., William, J.D., 1992. "Chlorine residual distribution in a municipal water network". *NEWWA Computer Symposium*, pp. 1-20.
- Gad, A. M., and Mohamed, H. I. (2014). "Impact of pipes networks simplification on water hammer phenomenon", *Sadhana Journal*, 39 (5), 1227-1244.
- Ghimire, S. R.; Barkdoll, B. D., (2007). "Issues in energy consumption by municipal drinking water distribution systems", *World Environmental and Water Resources Congress, ASCE 2007*.
- Harding, B. L. and Walski, T.M. 2000. "Long time-series simulation of water quality in distribution system". *J. Water Resour. Plng. And Mgmt*, 126(4), pp. 199-209.
- Kumarasamy, M. V.; Naicker, S.; Adu, J. T.; and Ocholla, P. O. (2017). "An analysis on optimization of domestic water conservation practices", *Int. Jour. Of Applied Eng. Research*, ISSN 0973- 4562, 12 (24), pp. 14958-14963.
- Martinez-Solano, J.; Iglesias-Rey, P. L.; Perez-Garcia, R.; Lopez-Jimenez, P. A. (2008). "Hydraulic analysis of peak demand in looped water distribution network", *Jour. of Water Resour. Planning & Management, ASCE*, Vol. 134, No. 6, pp. 504-510.
- Mohamed, H. I., and Abozeid, G. (2011). "Dynamic simulation of pressure head and chlorine concentration in the city of Asyut water supply network in abnormal operation conditions", *Arabian Journal for Science and Engineering*, 36 (2), 173-184.
- Mohamed, H. I., Eldardir, M. S., and Abdel-Hafiz, E. A. (2019). "Effect of roof tanks filling on the hydraulic performance of water supply pipes network", *Jour. of Eng. Science, Assiut University*, 47 (3), 280-295.
- Molino, B.; Rasulo, G.; Tagliatalata, L. (1991). "Peak coefficients of household potable water supply", *Water Resour. Managm.*, 4, pp. 283-291.

- Notter, R.H. and Sleicher, C.A. 1971. "The eddy diffusivity in the turbulent boundary layer near a wall". *Chem. Eng. Sci.*, 26, pp. 161-171.
- Ozdemir, O.N. 2005. "Success of booster chlorination for water supply networks with genetic algorithms". *Jour. of Hydraulic Research*, Vol. 43, No. 3, pp. 267-275.
- Roshani, E., and Filion, Y. (2014), "The effect of future water demand reduction on WDS rehabilitation planning", *Proc. Eng.*, 89, pp: 796-803.
- Rossman, L.A., Boulos, P.F. and Altman, T. 1993. "Discrete volume-element method for network water-quality models". *J. Water Resour. Plng. And Mgmt*, 119(5), pp. 505-517.
- Rossman, L.A. 1994. "EPANET users manual". Risk reduction engineering lab, U.S. Environmental protection agency, Cincinnati, OH.
- Rossman, L.A., Clark, R.M. and Grayman, W.M. 1994. "Modelling chlorine residuals in drinking-water distribution systems". *J. Env. Eng.*, 120(4), pp. 803-820.
- Rossman, L.A. and Boulos, P.F. 1996. "Numerical methods for modelling water quality in distribution systems: A comparison". *J. Water Resour. Plng. and Mgmt*, 122(2), pp. 137-146.
- Rossman, L.A. 2000. "Computer models/ EPANET". *Water distribution systems Handbook* (edited by L. W. Mays), McGraw-Hill, New York.
- Salgado, R., Tonidi, E. and O'Connell, P.E. 1988. "Extending the gradient method to include pressure regulating valves in pipe networks". *Proc. Inter. Symposium on Computer Modelling of Water Distribution Systems*. University of Kentucky, May 12-13.
- Sapkota, M., Arora, M., Malano, H., Moglia, M., Sharma, A., George, B. and Pamminger, F., (2015), "An overview of hybrid water supply systems in the context of urban water management: challenges and opportunities", *Water*, 7, pp: 153-174.
- Speight, V. L. (2014). "Impact of pipe roughness on pumping energy in complex distribution systems", *Procedia Engineering*, 70, pp. 1575-1581.