

See discussions, stats, and author profiles for this publication at:
<https://www.researchgate.net/publication/270049008>

Hydraulic modelling of water supply distribution for improving its quantity and quality

Article *in* Sustainable Environment Research · January 2013

CITATIONS

7

READS

236

4 authors, including:



Mohamed Elsamadony

Tanta University

11 PUBLICATIONS **97** CITATIONS

[SEE PROFILE](#)



Hazem Saleh

Minoufiya University

24 PUBLICATIONS **136** CITATIONS

[SEE PROFILE](#)



Ibrahim Rashwan

Tanta University

31 PUBLICATIONS **53** CITATIONS

[SEE PROFILE](#)

Some of the authors of this publication are also working on these related projects:



Isotope hydrology and water resources [View project](#)



Hydraulic Jump in Circular Open Channels [View project](#)

All content following this page was uploaded by [Mohamed Elsamadony](#) on 27 December 2014.

The user has requested enhancement of the downloaded file.

Hydraulic modelling of water supply distribution for improving its quantity and quality

Mahmoud A. Elsheikh,^{1,*} Hazem I. Saleh,¹ Ibrahim M. Rashwan² and
Mohammed M. El-Samadoni²

¹Department of Civil Engineering
Menoufia University
Shebin Elkom 32511, Egypt

²Department of Civil Engineering
Tanta University
Tanta 31512, Egypt

Key Words: Water distribution network, calibration, residual chlorine, optimal design

ABSTRACT

Large Egyptian cities are characterized by having old water distribution networks upon which new extensions are linked in order to serve new areas, adding complexity to an aging network where full rehabilitation may not be practical due to economic constraints. In this research, the water distribution network of Tanta City, one of the major Egyptian cities, was studied for optimizing its design and extension taking into consideration the effect of aging of the network and the most practical approach for its future configuration. A Water CAD Haestad software for hydraulic model of the network was carried out backed with field data and tests to check the main parameters of concern of the model for calibration and further use for future extension of the network. System optimization was carried considering main and secondary loops that included 588 pipes. Calibration was conducted through three types of readings namely static pressure, fire flow tests and extended period readings. Calibration of residual chlorine has been done on the network after hydraulic calibration by determining bulk and wall reactions. After calibration, optimal design has been determined for the model through defining an objective function of minimum cost design satisfying all constraints entered to the program considering coming plans for next 10 yr.

INTRODUCTION

Water distribution systems that are vital in supplying communities with their water demand include pipes, pumps, valves and reservoirs and are designed to deliver water at adequate discharge and pressures according to demands and with specific quality. Their cost represent 60-80% of the overall water supply system thus the importance of an optimized approach for design with special consideration to financial aspects including initial cost and energy consumption.

Hydraulic modeling provides an effective tool in this regard where the hydraulic calibration can be defined as the process of comparing a model results to field observations. Walski [1] discussed techniques for calibrating water distribution model by determining pipe roughness and water use. Ormsbee

and Lingireddy [2] described 7 steps process for model calibration, including: identifying model use, determining parameter estimates, collection of calibration data, evaluation of model results, macro calibration, sensitivity analysis, and micro calibration. Pipe grouping for roughness calibration was based on pipe type, age, and size. The optimization approach for calibration was compared to the analytical and simulation approaches based on skeletonizing of the system by removing pipes. Walski [1] suggested that pressure-measuring devices should be located near points of high demand, near the perimeter of the skeletonized network, and generally distant from water sources. De Schatzen [3] used genetic algorithms to calibrate a hydraulic model using data from pressure loggers. The average absolute error of the model was compared to the placement density of the pressure loggers. He showed that the average

*Corresponding author
Email: mshafy2@yahoo.com

absolute error in the model decreased as more data loggers were used. ATSDR [4] illustrated that an average pressure difference of ± 15.2 kPa (± 1.51 m) with a maximum difference of ± 50.3 kPa (± 5.03 m) represents a "Good" data set and an average pressure difference of ± 29.6 kPa (± 2.96 m) with a maximum difference of ± 97.9 kPa (± 9.79 m) represents a "Poor" data set. Pasha and Lansley [5] studied the effect of parameter uncertainty on water quality in a distribution system under steady and unsteady conditions that were analyzed using Monte Carlo simulation. The study provided guidance on difficulties in model calibration. For example, the wall decay had the largest influence on model prediction for the system that was reviewed and is one of the most difficult parameters to measure given its variability between pipes. Baggett et al. [6] illustrate that models generally incorporate a calibrated wall reaction rate coefficient with initial estimates based upon pipe roughness coefficients, flow velocity and pipe diameter. Yang et al. [7] aimed to supply good quality water to customers within specific pressure levels under various demand conditions. Elsheikh and Basiouny [8] used approaches of modelling water distribution networks (WDNs) to characterize the main aspects of water quality deterioration in a distribution system and the effect of residence time on chlorine uptake and the formation and evolution of disinfection by-products.

van Dijk et al. [9] used model to optimize the water distribution backbone of a city using pipes, nodes, and reservoirs. In this model the pipes take up a central position as it is the selection of diameter sizes for pipe segments that governs the optimization process. Smaller diameters are less expensive to procure, while larger diameters result in higher water pressure at the demand nodes in the network. The difficulty of optimizing water distribution systems is mainly due to the discrete nature of the decision variables and the size of the search space, which can be calculated as the number of possible discrete pipe diameters (the available commercial diameters) to the power of the number of pipes in the network.

Chong and Zak [10] described the measure of goodness of the alternatives described by an objective function or performance index. Optimization theory and methods address the selection of the best alternative through the given objective function. Bhave [11] illustrates that the design of a WDN for a large water distribution system is a complex problem and involves decisions on pipe layout and sizes; location and capacity of tanks; location, types, capacity, and operating schedule of pumps; and location, types, and settings of different valves. Mays and Tung [12] showed that the cost of the system includes the initial investment for the components, such as pipes, tanks, valves and pumps, and the operating cost for pumping the water throughout the

system. Mays [13] determined that the main constraints are the desirable demands to be supplied with adequate pressure at withdrawal locations. Also, the flow of water in a distribution network and the pressure heads at nodes must satisfy the governing laws of conservation of mass and energy. The WDN design problem can be stated as to minimize (Initial Capital Cost + Operations Cost) and subjected to conservation laws of mass and energy, water demand constraints and meet nodal head requirements.

WaterCAD User's Guide [14] shows that Darwin Designer uses a genetic algorithm generic search paradigm to help hydraulic engineers efficiently plan and design a water distribution system. Zheng and Walski [15] illustrate that there are three types of optimization models including least cost design, maximum benefit design, and cost-benefit tradeoff design. Least cost optimization searches for the optimal solution by minimizing the cost while satisfying the design constraints.

The main objective of the present study is to study the hydraulic behavior of Tanta City's WDN through developing appropriate hydraulic model representing the main and selected secondary pipes arising to 588 pipes as to simulate the hydraulic performance of this network constructed decades ago. Calibration of residual chlorine representing one of the main indicators of water quality within network is also conducted after hydraulic calibration through determining bulk and wall reactions. After calibration, optimal design has been determined for the model through defining an objective function of minimum cost design satisfying constraints and considering future requirements.

MATERIALS AND METHODS

1. WDN of Tanta City

The potable WDN under study serves Tanta City, the capital of El-Gharbiya Governorate Egypt and has been in service since 1924. The network is fed through both surface water (from three plants producing $67,200 \text{ m}^3 \text{ d}^{-1}$) and ground water (from ten wells sites producing $92,300 \text{ m}^3 \text{ d}^{-1}$) with total water production for the city of $159,500 \text{ m}^3 \text{ d}^{-1}$.

1.1. Model skeletonization

In order to skeletonize the model to an acceptable degree, the study adopted US EPA [16] conditions used as follows: (1) At least 50% of total pipe length in the distribution system (as shown in Table 1 for current study reaching 51%); (2) At least 75% of the pipe volume in the distribution system (as shown in Table 1 for current study reaching 86%); (3) All 300 mm diameter and larger pipes; (4) All 200 mm and larger pipes that connect pressure zones, influence zones from different sources, storage facilities, major

demand areas, pumps, and control valves, or are known or expected to be significant conveyors of water; (5) All 150 mm and larger pipes that connect remote areas of a distribution system to the main portion of the system; (5) All storage facilities with controls or settings applied to govern the open/closed status of the facility that reflects standard operations; and (6) All active pump stations with realistic controls or settings applied to govern their on/off status that reflects standard operations.

1.2. Allocating demands

Water CAD Haestad software [17] was used for the hydraulic analysis of Tanta WDN. From analysis of readings of the data collected for the period from 2010 to 2011, it was found that water consumption per capita reached 296 L d⁻¹ in 2011 with the demand pattern shown in Fig. 1.

Unaccounted water was estimated by calculating the difference between the total supply from the billing records and the system's daily supply for the same period. The method for allocating baseline demands is flow distribution method illustrated in Water CAD Haestad Methods [17] that involve distributing a lump-sum demand among a number of service polygons.

1.3. Skeletonized WDN

Tanta City was divided into twelve administrative divisions as shown in Fig. 2 serving a population of 322,700 at the end of 2011. The skeletonized network following the above mentioned skeletonization procedure consists of 588 pipes with diameters of 100, 150, 200, 250, 300, 350, 400 and 600 mm. The modeled pipes were fabricated from four different pipe materials namely Cast Iron installed in 1924, Asbestos Cement (AC) installed in 1960, Ductile Iron installed in 1980 and Polyvinylchloride installed in 2000.

Developing the hydraulic model for Tanta City network involved the following steps: (1) Creating a pipe network from the water system GIS files using WaterGEMS (Bentley Systems) and ArcGIS (Environmental Systems Research Institute); (2) Spatially allocating customer demands and pipe leakage to pipe network nodes; (3) Assigning elevations to pipe network nodes; (4) Incorporating boundary condition elements (e.g., pumps, ground storage reservoirs); and (5) Performing calibration and a quality control review of the resulting model.

2. Calibration

2.1. Calibration data

Devices used for taking field measurements were the static pressure gauges and flow measurement device by Ultra-Sonic waves to take three types of data as follows: (1) steady state measurements; (2) fire flow tests; and (3) extended period measurements

The hydraulic model was calibrated by adjusting Hazen-Williams coefficients (C) according to these three types of field measurements data:

- Thirty-six fire hydrants distributed on the network of Tanta City were selected. Then the pressure was measured to give a good representation of the pressure in the network. The entire field measured pressures were taken during the maximum hourly demand scenario.
- Twelve tests for the fire hydrants were carried out in the time of maximum summer consumption during the period of July 25, to July 31, 2011.
- Two fire hydrants were determined to record pressure readings every hour for 24 h continuously; these were H5 and H35 as there are no elevated tanks connected to the network of the city. The two fire hydrants were selected as far as possible from water treatment plants.

2.2. Calibration statistics

There are many ways to judge on the performance of model calibration, the calibration statistics used in this study was by calculating the squared relative difference between observed and simulated pressure for each test. The results and the observation data were entered to an Excel sheet and the value of squared error was calculated for every test then the mean square error and standard deviation calculated from Excel sheet: the lower values of these parameters, the higher is the accuracy of the calibration process.

2.3. Calibration process

The C-factors for calibrated model pipes were found to be as follows:

C-factor for AC pipes (60% of the network) was around 53 as calculated from some tests and an average value has been taken. For the rest of pipes the aged C-factor calculated from Eq. 1.

$$C = 18 - 37.2 \log \left(\frac{\varepsilon_o + at}{D} \right) \quad (1)$$

Table 1. Length and volume of pipes in the model before and after skeletonization

	Before model skeletonization	After model skeletonization	Percentage of skeletonization
Total length of pipes	206488 m	105523 m	51.1%
Total volume of pipes	6034.1 m ³	5197.9 m ³	86.1%

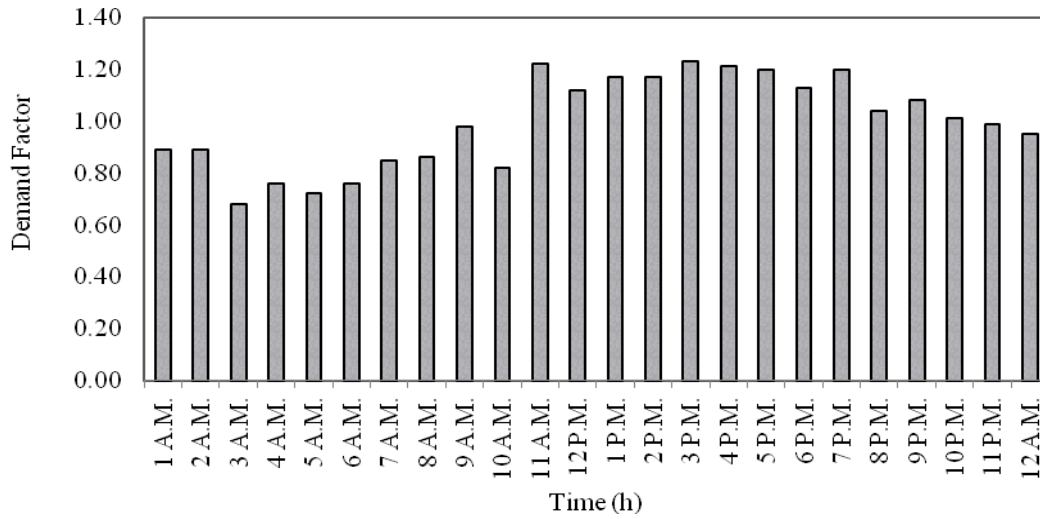


Fig. 1. 24-h composite demand pattern for different zones in Tanta City.

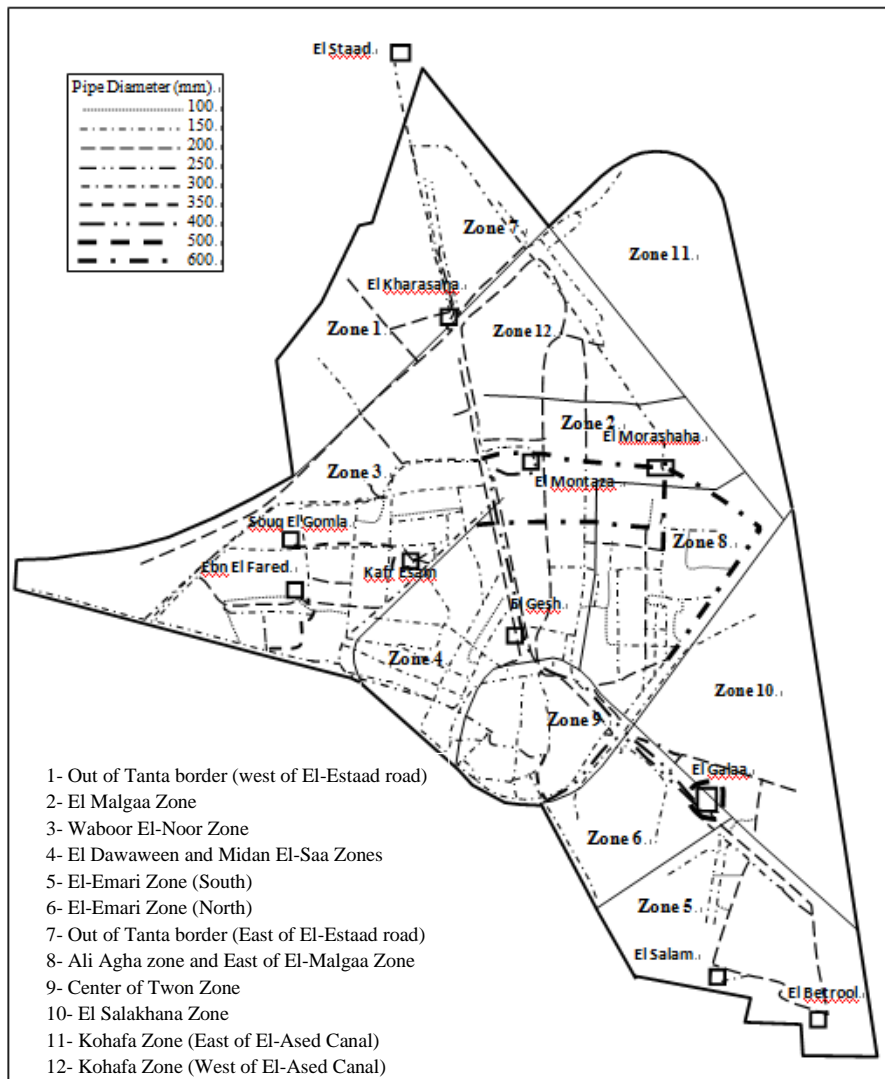


Fig. 2. Layout of administrative divisions and water distribution network for Tanta City.

Where: ϵ_0 = roughness height when pipe was new ($t = 0$) (mm), a = rate of change in roughness height (mm yr^{-1}), t = age of pipe (yr) and D = diameter of pipe (mm).

The pipes were divided into five groups according to their aged C-factors. Each group was given a range as shown in Table 2 with incremental steps of 5 till reaching the calibrated value (C_{taken}) which gave the best results. Pressure correlation of field data and simulation results agreed well, according to Walski [1].

RESULTS AND DISCUSSION

Calibrated model for the steady state field measurements network gave results that matched well with measured data (Fig. 3) as the mean square error was 0.00814. Values of 52.6% of the simulated values of static pressure resulted in absolute pressure difference of less than 13,8 kPa and 84.5% of the simulated values of static pressure resulted in absolute pressure difference of less than 34.5 kPa. The maximum absolute pressure difference was 47.6 kPa and average pressure difference of 17.9 kPa.

Also, the calibrated model for the fire flow tests gives an average absolute pressure drop due to open hydrants difference between measured and simulated results equal 6.7 kPa with maximum absolute pressure drop difference equal 13.1 kPa. Moreover, the calibrated model for the extended period simulation gives an absolute pressure difference between measured and simulated pressure at H5 equal 33.8 kPa and at H35 equal 26.2 kPa (Fig. 4).

1. Total Chlorine Residual Modelling

Computer-aided simulations considerably improve understanding of the fate and transport of disinfection chemicals, thereby offering a tool to improve treatment practices and ultimately lower overall operation costs. Nevertheless, to ensure the accuracy of the model, it is critical that the constituent reactions and decay/growth mechanisms on which the model is based are sound and appropriate.

To calibrate the model for predicting disinfectant residual concentration, chlorine degradation rates were determined by collecting field samples as well as conducting laboratory "bottle tests". The data were

Table 2. Pipes roughness group

Group	Range	C_{taken}
Group 1	(70-100)	90
Group 2	(80-110)	100
Group 3	(90-120)	110
Group 4	(100-130)	120
Group 5	(110-140)	130

then analyzed to understand those reactions occurring in the main portion of the stream flow (bulk reactions), as well as those occurring on or near the pipe wall (wall reactions).

1.1. Bulk reaction coefficients

Bulk reactions are reactions that are unaffected by processes involving the pipe walls and can be described by n^{th} order kinetics [18]:

$$R(C) = k_b C^n \quad (2)$$

Where: R = reaction rate, C = reactant concentration, k_b = bulk reaction rate coefficient and n = reaction order.

Chlorine decay is usually represented as first order (i.e., $n = 1$), with the decay coefficients typically being between 0.05 and 15 d^{-1} ; noting that coefficients for decay reactions can be reported as negative values. The site-specific decay coefficient is typically determined using bottle tests where chlorine decay in a particular volume of water is monitored over the natural maximum water age of the system as it has been done on the six stations which injected chlorine to the network within the study area. The chlorine values of the finished water are the target chlorine concentrations for determining the chlorine dosage to the filtered water in a water treatment plant [19].

From Fig. 5 the average k_b value taken for all stations was -0.032 h^{-1} where the WaterGEMS[®] accepts only one value for each constituent like "chlorine".

1.2. Wall reaction coefficients

Wall reactions depend on the bulk conditions, pipe dimensions (i.e., pipes with smaller diameters encourage greater solution/wall interaction and therefore greater reaction rates), and pipe wall condition:

$$R(C) = \left(\frac{A}{V}\right) K_w C^n \quad (3)$$

Where: K_w = wall reaction rate coefficient and (A/V) = surface area per unit volume within a pipe.

The dependency of K_w and the reaction order on pipe material and condition (i.e., age, encrustation and corrosion) makes determining the coefficients difficult. Although conceptually K_w may be measured under ideal conditions (i.e., long isolated pipes, no connections, controlled flow, inline chlorine measurements), in real-world conditions such measurements are infeasible [17]; therefore, models generally incorporate a calibrated K_w with initial esti-

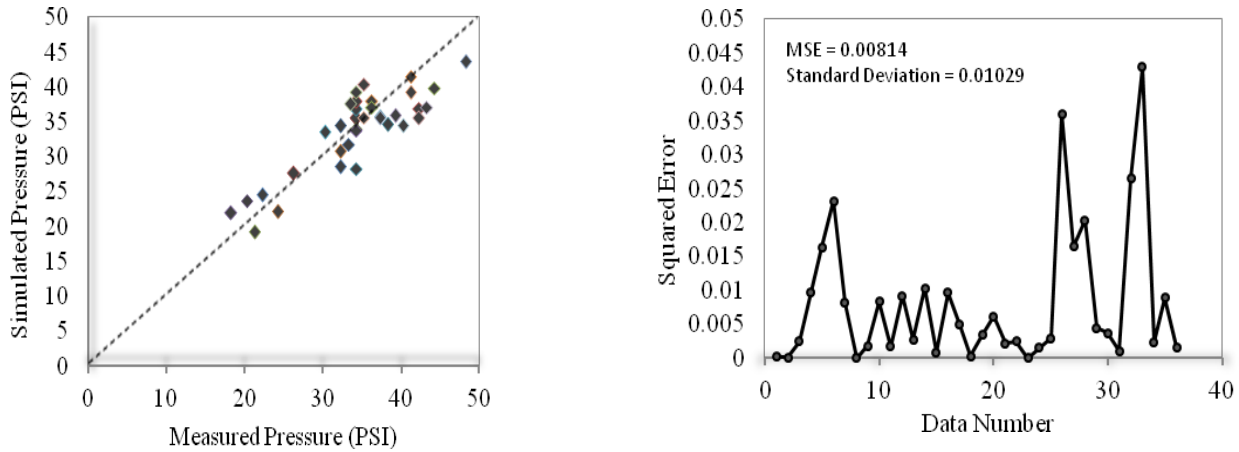


Fig. 3. Relation between measured pressure and simulated pressure in calibrated model. *The pressure is measured in PSI which is equal to 6.895 kPa.

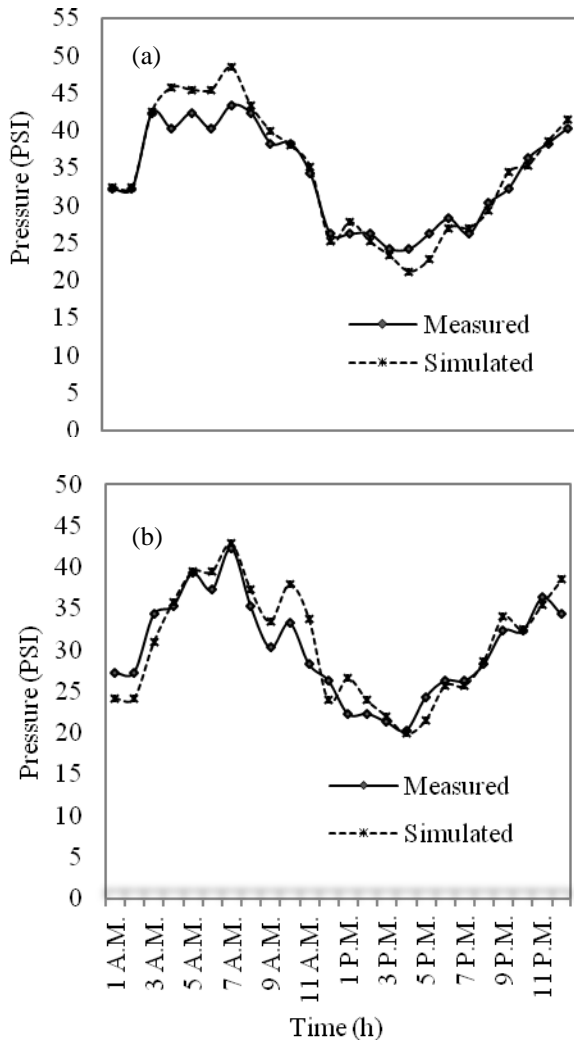


Fig. 4. Measured and simulated pressure during the day in calibrated model (a) at H5 and (b) at H35. *The pressure is measured in PSI which is equal to 6.895 kPa.

mates based on pipe roughness coefficients, flow velocity, and pipe diameter, as demonstrated by Eq. 4:

$$K_w = \left(\frac{\alpha}{C_c} \right) \tag{4}$$

Where: α = correlation coefficients of wall reaction and pipe roughness and C_c = calibrated C.

This approach is practiced widely in the industry because wall decay coefficients vary greatly due to pipe condition (material, roughness, corrosion, and biofilms) and cannot be measured reasonably for large distribution systems. Wall decay coefficients were assigned using α of -38.5, and the hydraulically calibrated C (Table 3).

1.3. Total chlorine residual modeling results

Figure 6 illustrate the typical correlation between field and model data observed at point 2 of the WDN. The simulated total chlorine concentrations at these select sites are in good agreement with the field data. The model provides practical value by reasonably predicting locations of residual chlorine concentration at all junctions of the network all over the day. The minimum concentration is 0.13 mg L⁻¹ while the maximum concentration is 3.30 mg L⁻¹. Many types of chlorination forms are used in the system. It is recommended to use chlorine dioxide and not sodium hypochlorite because chlorine dioxide is more effective and clearly safe [20,21].

2. Conservative Parameters Simulation

After hydraulic calibration, concentrations of different conservative parameters of water sources were introduced to the model in order to obtain the concentration of those elements in network. It was found that there is a zone within the network where

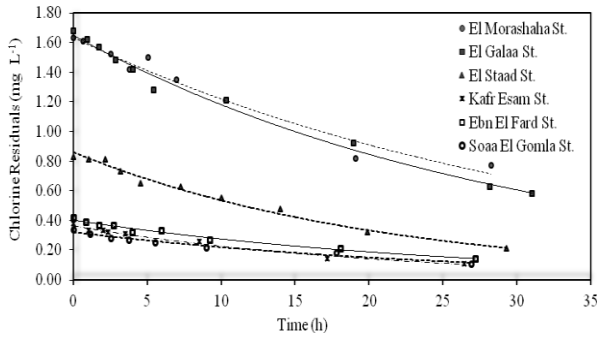


Fig. 5. Bulk phase dissipation curve for each station.

Table 3. Values of wall reaction rate coefficient (Kw) for different C-Coefficients

Coefficients used in calculating Kw*	
Hazen-Williams "C" Coefficient	Kw (ft d ⁻¹) - α = -38.5
53	-0.276
90	-0.428
100	-0.385
110	-0.35
120	-0.321
130	-0.296

*The coefficients Kw is calculated in ft d⁻¹ which is equal to 0.305 m d⁻¹.

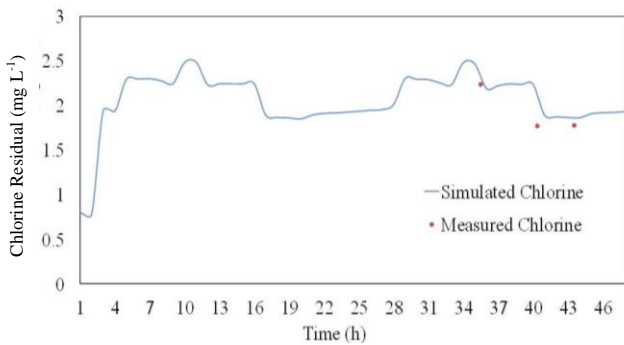


Fig. 6. Measured and simulated chlorine residual at point (2).

concentrations of iron and manganese are higher than admissible values according to Egyptian guidelines amounting to 1 mg L⁻¹ for iron and 0.5 mg L⁻¹ for manganese. The reason of these high concentrations is due to the existence of some wells located in this zone with high concentrations of Iron and Manganese.

3. Mathematical Formulation for Optimization

With carefully defined design objectives and design constraints, water distribution optimization models of the least cost design, maximum benefit design, and cost-benefit tradeoff design are generalized as a constrained nonlinear programming problem, that is to search for the optimal design

solution (design variables), subject to a set of nonlinear constraints, given as:

3.1. Objective function

The used objective function, in case of least cost design, will be formulated by entering the cost factor in the optimal design and this objective function was as following:

$$\text{Minimize } \left(\sum_{k=1}^{DP} C_k(d_k)L_k + \sum_{k=1}^{RP} C_k(d_k, e_k)L_k \right) \quad (5)$$

Where: C_k(d_k) = cost of design pipes for each diameter d_k per unit length of a pipe, C_k(d_k, e_k) = cost of rehabilitation pipes for each diameter d_k and each rehabilitation action e_k per unit length of a pipe, L_k = length of the k pipes, DP = number of design pipes and RP = number of rehabilitation pipes.

3.2. Constraints

Constraints can be classified into three groups, implicit bound constraints, explicit bound constraints and implicit system constraints.

3.2.1. Implicit bound constraints

1. Satisfy the minimum required junction pressure of 138 kPa and the maximum allowable pressure of 690 kPa.
2. Satisfy the maximum flow velocity of 1.8 m s⁻¹ for distribution pipes and 3.1 m s⁻¹ for transmission mains.

3.2.2. Explicit bound constraints

These constraints are used to set limits on explicit decision variables of the optimization problem; there are a set of commercially available pipe diameters. The minimum and maximum diameters were assigned to each design variable group to choose from this set the solution which narrows the scope of the optimization which ensures the minimum cost design taking into account these constraints.

3.2.3. Implicit system constraint

Include nodal conservation of mass and conservation of energy. Mathematically, these constraints can be expressed as follows:

1. Nodal conservation of mass: inflow and outflow must be balanced at each junction node as follows:

$$\sum Q_{in} - \sum Q_{out} = Q_e \quad (6)$$

Where: Q_{in} = flow into the junction, Q_{out} = flow out of the junction and Q_e = external inflow or demand at the junction node.

2. Conservation of energy: head loss around a closed loop must equal zero or pump energy head if there

is a pump.

$$\sum h_f = \text{zero (around each loop in case of there is no pump)} \quad (7)$$

$$\sum h_f = E_p \quad (\text{If there is a pump}) \quad (8)$$

Where: h_f = head loss due to friction in a pipe and E_p = the energy supplied by a pump.

3.3. Optimization model

The future application of the calibrated models has to be provided to strengthen Tanta WDN for supplying water to consumers in sufficient quantities, pressure and quality. Optimized hydraulic model for the studied water distribution system was developed considering current and future plans (in 2022) by using Darwin Designer, an integrated optimization modeling tool for efficiently evaluating design solutions. The tool allows modelers to set up the multiple design criteria and conduct optimization design runs for different loading conditions, boundary settings (tank level, pump/pipe status and valve setting), design pipe groups and hydraulic constraints.

In this model, design variables has been defined to the program with available pipe sizes of 100, 150, 200, 250, 300, 350 and 400 mm for old pipes plus pipe size equal 0 mm for new pipe connections. A zero size is to represent the option that there is no need to lay a new pipe.

Unitized benefit function has been calculated to know the benefit of the design and network rehabilitation. The unitized benefit function can be expressed as an average increase in pressure and flow in the entire network. This function can be simplified as following:

$$P_{avg} = \frac{\sum_{j=1}^{JN} |P_j| - |P_{min}|}{JN} \quad (9)$$

Where: P_{avg} = average increase in pressure, P_j = pressure at junction j and JN = number of junctions.

This function can evaluate the enhancement in the pressure and flow for each trial. The average pressure increase means that the average of difference between the junctions pressure and the required minimum pressure (138 kPa). The value of the average increase in pressure for the current status of the network was equal to 78 kPa.

4. Optimizing the Studied Network Considering Current Status (2012)

The optimized design involved changing many pipes. The length of changed pipes was 12,782 m, i.e., 12.1% of total length of the network and the changed volume (capacity) is 412.44 m³, or 7.9% of total

capacity of the network. This solution resulted an average pressure increase (P_{avg}) equal to 120 kPa which was higher than the current status by 42 kPa.

4.1. Water age of optimal design system at year 2012

The junction subject to the highest water age of the system was (J-14) and its value was 10.55 h. After applying the optimal design, water age of the system at the same junction increased up to 11.94 h. Increase in water age of the system results from increased diameters which led to increase network's capacity to carry more quantity of water with fixed demand loading.

4.2. Residual chlorine in optimal design at year 2012

The effect of the optimal design on residual chlorine has been investigated. There was good distribution of residual chlorine after applying the optimal design at the maximum hourly demand (3:00 pm). Residual chlorine increased after applying optimal design according to proposed enhancements as a result of decreased wall reaction of these pipes so that the residual chlorine increased in the entire network.

CONCLUSIONS

The result of the current study for modeling and optimizing one of the oldest yet operating WDN in large Egyptian city of Tanta addressing both hydraulic aspects and water quality can be summarized as follows:

- Estimates of average water consumption per capita that reached 296 L d⁻¹ in 2011 with 40% unaccounted water.
- Calibration activity of the hydraulic model showed a reduced C-factor for AC pipes to 53 (very low compared with normal condition). This could be because of the high age of installed pipes in the network.
- Proposed enhancements were quantified and financially analyzed, with proposed replacement of 12.1 and 29.8% of total length of the network to cope with current and future needs respectively.
- Water quality within the distribution network showed high concentrations of iron and manganese in areas nearby wells and high chlorine decay in the networks. Recommendations were derived from the study for specific locations of re-chlorination doses and mixing percentage of well and surface water.

REFERENCES

1. Walski, T.M., Technique for calibrating network models. J. Water Res. Pl.-ASCE, 109(4), 360-372 (1983).
2. Ormsbee, L.E. and S. Lingireddy, Calibrating hydraulic network models. J. Am. Water Works

- Ass., 89(2), 42-50 (1997).
3. De Schaetzen, W., Optimal Calibration and Sampling Design for Hydraulic Network Models. Ph.D. Dissertation, School of Engineering and Computer Science, University of Exeter, Exeter, UK (2000).
 4. ATSDR, Analysis of the 1998 Water-Distribution System Serving the Dover Township Area, New Jersey. Agency for Toxic Substances and Disease Registry, Atlanta, GA (2000).
 5. Pasha, M.F.K. and K. Lansley, Effect of parameter uncertainty on water quality predictions in distribution systems-case study. *J. Hydroinform.*, 12(1), 1-21 (2010).
 6. Baggett, C.C., G. Li, R.A. Rosario, A.Y. Khan, W.A. Lovins III, R.M. Powell and T. Reed, From start to finish: Calibrating Pinellas County's 2,000-mile hydraulic water distribution system model. *Florida Water Resour. J.*, December, 44-54 (2008).
 7. Yang, S., N. Hsu, P. Louie and W. Yeh, Water distribution network reliability: Connectivity analysis. *J. Infrastruct. Syst.*, 2(2), 54-64 (1996).
 8. Elsheikh, M.A.E. and M.E. Basiouny, Deposition and formation of THMs in water supply system. *Sustain. Environ. Res.*, 21(2), 89-94 (2011).
 9. van Dijk, M., S.J. van Vuuren and J.E. van Zyl, Optimising water distribution systems using a weighted penalty in a genetic algorithm. *Water SA*, 34(5), 537-548 (2008).
 10. Chong, E.K.P and S.H. Zak, An Introduction to Optimization. John Wiley & Sons, New York (2001).
 11. Bhave, P.R., Optimal Design of Water Distribution Networks. Alpha Science, New Delhi, India (2003).
 12. Mays, L.W. and Y.K. Tung, Hydrosystems Engineering and Management. Water Resources Publications, Highlands Ranch, CO (1992).
 13. Mays, L.W., Water Distribution System Handbook. McGraw Hill, New York (2000).
 14. WaterCAD User's Guide: Water Distribution Modeling Software. Haestad Methods, Waterbury, CT (1999-2003).
 15. Zheng, Y.W. and T.M. Walski, Optimizing water system improvement for a growing community. International Conference of Computing and Control in the Water Industry. Exeter, UK, Sep. 5-7 (2005).
 16. USEPA, Water Distribution System Analysis: Field Studies, Modeling and Management: A Reference Guide for Utilities. US Environmental Protection Agency, Washington, DC (2005).
 17. Haestad Methods, Walski, T.M., D.V. Chase, D.A. Savic, W.M. Grayman, S. Beckwith and E. Koelle, Advanced Water Distribution Modeling and Management. Haestad Press, Waterbury, CT (2003).
 18. Clark, R.M. and W.M. Grayman, Modeling Water Quality in Drinking Water Distribution Systems. American Water Works Association, Denver, CO (1998).
 19. Ahn, J.C., S.W. Lee, K.Y. Choi and J.Y. Koo, Application of EPANET for the determination of chlorine dose and prediction of THMs in a water distribution system. *Sustain. Environ. Res.*, 22(1), 31-38 (2012).
 20. Hsu, C.S., I.M. Chen and D.J. Huang, Quality investigation and disinfection of spring waters with different disinfectants. *Sustain. Environ. Res.*, 22(6), 357-361 (2012).
 21. Hsu, C.S., W.Z. Huang and H.Y. Wang, Evaluation of disinfection efficiency between sodium hypochlorite and chlorine dioxide on spa water. *Sustain. Environ. Res.*, 21(6), 347-351 (2011).
-
- Discussions of this paper may appear in the discussion section of a future issue. All discussions should be submitted to the Editor-in-Chief within six months of publication.
- Manuscript Received: January 31, 2013**
Revision Received: April 8, 2013
and Accepted: May 13, 2013