BULK CHLORINATION DECAY MODELING FOR BENGHAZI WATER DISTRIBUTION SYSTEM

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Abstract

Quality of water is often measured by the residual amount of chlorine in a distribution system. Therefore, it is necessary to define the decay process of the disinfectant agent to come up with initial dose of chlorine. The big challenge facing the most of water utilities is how to maintain concentration of chlorine within recommended range (0.2 – 0.5 mg/l) in whole system and all time. Therefore, determining a required initial concentration of chlorine that should be given to a system to maintain chlorine concentration within allowable limits throughout whole system is not an easy task. This paper aimed to determine the initial concentration of chlorine that must be pumped into the Benghazi water network in order to maintain its quality. To achieve this goal, a hydraulic model of the network was constructed using the EPANT simulator. Where all elements of the network were represented, including the age of the pipes. Accordingly the optimal initial concentration of chlorine was determined (4 mg/l), which gives residual values of chlorine close to the values obtained from the laboratory, and by plotting the logarithm of these values with the logarithm of the optimal initial concentration, the decay rate was found equal to -0.055 1/day.

Keywords: bulk decay, chlorination, water distribution system.

الملخص

تهدف هذه الورقة الى تحديد الجرعة او القيمة الابتدائية من الكلور التي يجب ضخها في شبكة مياه بنغازي للمحافظة على جودة المياه فيها. ومن اجل ذلك تم انشاء نموذج هيدروليكي باستخدام برنامج المحاكاة Epanet حيث تم تمثيل ومحاكاة كل عناصر الشبكة بما فيها عمر الانابيب. عليه تم ايجاد القيمة الافضل للجرعة الابتدائية المطلوبة من الكلور ، والتي بلغت 4 ملغم / لتر، والتى ينتج عنها قيم للجرعة المتبقية قريبة جدا من القيم التى تم تحديدها في المعمل . وبرسم هذه القيم على مقياس لوغرثمي ضد القيم الافضل (المثلى) من الجرعة الابتدائية لفترات زمنية مختلفة تم تحديد معدل الاضمحلال للكلور في شبكة المياه ببنغازي والذي بلغ -0.055 لتر/ يوم.

1. INTRODUCTION:

Water quality during distribution is prone to deterioration due to several factors causing a possible health risk to consumers, and thus disinfection process becomes necessary. Disinfection in potable water treatment may be defined as reduction of pathogenic organisms to prevent waterborne diseases, (Brown et al., 2011). Chlorine is the most commonly applied disinfecting agent used worldwide to provide microbiologically safe drinking water. Therefore, it is important to maintain adequate chlorine residual in a distribution system for this purpose. According to the (WHO, 2008) concentration of chlorine in a distribution network should remain between 0.2 and 0.5 mg/l. However (Sarbatly, R. HJ. et al., 2007) in his study case in Malaysia reported that drinking water in warm countries should be treated as a unique case because of the climate. They found, the temperature around 25 – 35 C^o is quite optimal for most microbes to grow and it also increases chlorine evaporation rate that reduces agent acid in equilibrium reaction. Nevertheless, they did not introduce alternative limits. The big challenge facing the most of water utilities is how to maintain concentration of chlorine within recommended range in whole system and all time. Since, decay of any disinfectant agent, such as chlorine, is inevitable. Because chlorine beside its disinfection function, it reacts with both organic and inorganic substances which exist in water and thus readily to decay. As both organic and inorganic substances are exist in different concentrations and degrees of reactivity, loss of chlorine over time is a gradual process and the half-life of chlorine in treated water can vary from several hours to several days (Clark R.M et al., 2000). Non-organic materials like iron, manganese, sulphide, bromide and ammonia are reactive with chlorine strongly. These reactions are fast and occurring in seconds, (Brown et al., 2011). However, reactions of chlorine with organic matters make up majority of chlorine demand (Clark 1998) according to oxidizing characteristics of reactions, chlorine is prone to decay due to its organic reaction with bulk water and inorganic reaction with pipe material and attached bio-film. The decay of chlorine within bulk water is referred to as bulk decay of chlorine, while that due to bio-films and at distribution pipe wall are known as wall decay. Sum of the two processes is commonly termed as chlorine demand, , (Brown et al., 2011). The bulk decay , which is due to interactions with organic substances present in water depends mainly on hydraulic parameters of the flow. In low consumption periods the flow velocity is low giving thus time for the bulk reactions to occur (higher bulk decay rate) (Menaia et al., 2002) and (Sijia et al., 2008). In addition to the natural organic concentration (Brown et al., 2011) found that, temperature and initial chlorine concentration are significant in bulk decay. In terms of reaction significance, (Hallam et al., 2002) and (Silja et al., 2008) found that, in cast iron (CI) pipes wall decay is significant, while in polyethylene (PE), bulk decay is significant. From which, they conclude that, higher initial chlorine dose is required in case of CI pipes than in PE pipes to maintain acceptable level of chlorine in a system. Generally, Plastic pipe and relatively new lined iron pipe are not expected to exert any significant wall demand for disinfectants. Therefore, determining the required initial concentration of chlorine that should be given to a system to maintain chlorine concentration within allowable limits throughout whole system to maintain microbiological quality and minimize bio-film formation throughout the WDS is obliged. This paper presents an attempt to measure the chlorine bulk decay in Benghazi WDS as a step toward determining the optimal chlorine initial concentration that is supposed to be pumped into the network to keep the residual chlorine percentage in the network within the stipulated limits.

2. STUDY AREA

The model is applied for WDS of Benghazi. The water network of Benghazi (Fig. 1) consists of 418 segments of pipes, with a total length of 373.147 km and diameters varying from 150 to 2500 mm. A total of 36.4% of the pipes are 300 mm in diameter, and 27.4% are 400 mm in diameter; the other diameters are distributed as shown in Fig. 2. In term of materials, about 34% of the pipes are made of uncoated steel, and ductile iron pipes account for about 56% of the whole length; meanwhile, concrete pipes make up about 10%, as shown in Fig. 3. About 25% of the system is more than 36 years old, about 20% is 24 years old, and 30% is 5 years old; the rest of the system is about 27 years old on average, as shown in Fig. 4. The degradation of the network has made it unable to provide safe, potable water for domestic use, adequate quantities of water at sufficient pressure, which has resulted in major environmental and health problems in the city.

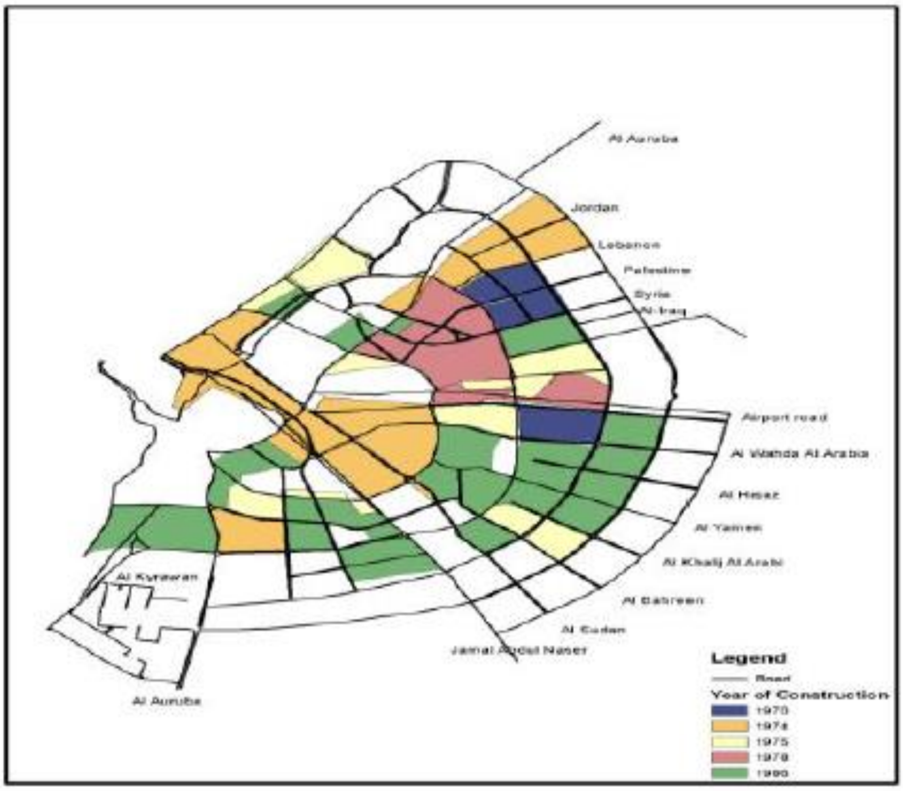
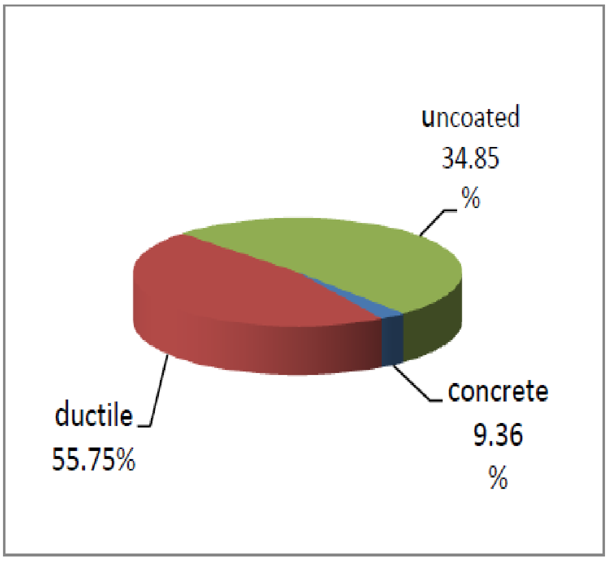


Fig 1 WDS of Benghazi



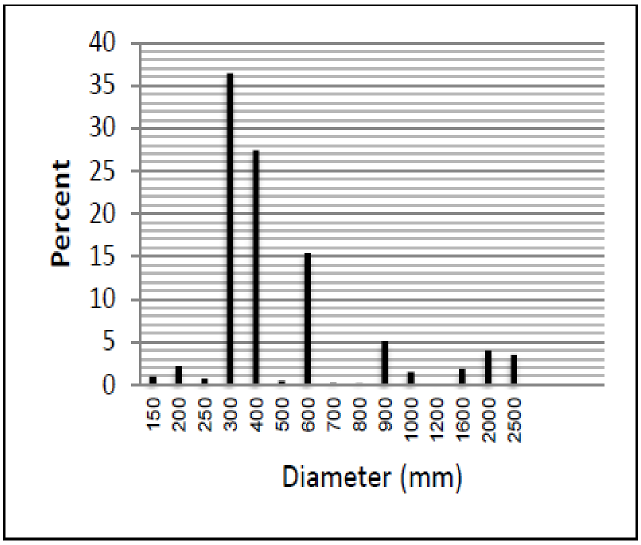


Fig 2 Percentage of diameters in WDS of Benghaz

Fig 3 percentage of pipes material in WDS of Benghazi

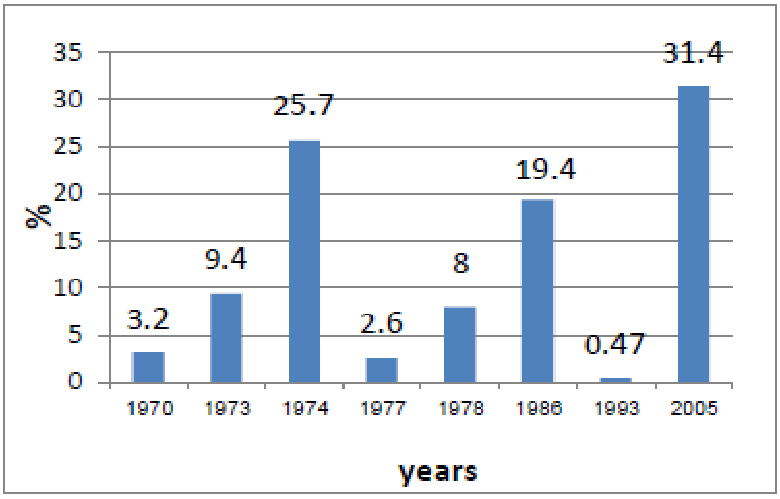


Fig 4 pipe age (year of installation)

3. THEORIES AND METHODOLOGY

Physically it is almost impossible to monitoring the decay of chlorine in WDS, alternatively, building a water quality model can provide a perfect key to this dilemma. However, solving dynamic models of water quality require that a calibrated hydraulic analysis must be preformed first to determine how flow volumes and directions change during time cycle of the simulation all through the system. In order to apply an extension period simulation (EPS) model, it is essential to define a set of operational rules; called “controls” that direct the model how the water system operates. By operational control status of flow or pressure, settings can be adjusted throughout the simulation to response predefined conditions. Pumps on/off, speed operations can be controlled in order to raise or lower pressure and flow rate, to response to predefined sets of water volume in tanks. When pressure and flow in a distribution system are variable during operation day, valves can be programmed to maintain allowed values of pressure and flow. For a pipe, the only status that can really vary is whether the pipe is open or closed. In order to determine whether a model represents the actual system, it is necessary to measure various system values, typically, pressure and flow, during field studies and then compare the field results with model outputs. This process is known as model testing. Adjustment and correction of the simulated model to match the actual system is known as a calibration process. Therefore, to calibrate a model, actual values of some system parameters must be measured directly from field. Characteristics that are typically set and adjusted include pipe roughness factors, minor losses, demands at nodes and decay rate (objective of this paper).

For the study system, not much data were found available, only pressures at some nodes are available (Table 1). These values were used to calibrate mainly pipe roughness in the model. A manual trial and error approach is used and a skeletonized system is produced, Fig 5 represents the hydraulic model for the study network.

Table 1 Comparison of measured and calibrated pressure at certain nodes dated on 15 Aug. 2020 at 14 o'clock. Source: Municipality of Benghazi.

Node	Actual pressure (m)	Calibrated pressure (m)
88	58.20	50.84
87	65.10	66.57
117	43.45	47.36
118	46.12	47.41
119	45.60	47.71
90	43.33	45.26

A skeletonized model denotes a model that includes only a major subset of actual pipes necessary to calibrate the model rather than all pipes in the network. The skeletonized process included: addition of key pipes, updates to consumer demand data, and an interconnection between the case study area and the full system by a fixed grade node (reservoir). Worth to mention, due to shortage of available data and the used extended period hydraulic simulation, calibration of the study system proved to be rather difficult. The roughness coefficients were adjusted to fit a network made of 0 to 45 years old , In this paper, the roughness-growth model proposed by (Sharp & Walski., 1988) is used to model the aging of pipes. This model predicts the temporal increase in roughness of a pipe resulting from processes such as internal corrosion, bio-film formation, and tuberculation. The model is dependent on the common concepts of head loss formulation like Hazen-Williams, that relate Hazen-Williams's C factor to time-varying relative roughness of a pipe (Eq. 1).

$$C = 18.0 - 37.2 \log X \quad , \quad \text{where } X = \frac{(e_0 + at)}{D}$$

(1)

Where e_0 = initial height of internal pipe roughness at time t=0 (mm);

a = growth rate in roughness height (mm/year);

t = age of a pipe;

D = pipe inner diameter (mm); and

X = time-varying relative roughness.

The literature for pipeline hydraulics were reviewed to find suitable values of primary roughness height e_0 and roughness growth rate "a". (Sharp & Walski., 1988) reported an initial roughness height e_0 for new steel pipe is 0.18 mm for sizes (150–600 mm). In addition, they performed a regression analysis using Eq.1 and data from (Lamont., 1981) and (Hudson., 1966) to find roughness growth rate a with relating to the corrosives of the water, those growth rate are shown in Table 2.

Table 2 Roughness growth rate a (mm/year) in literature

Researcher(s)	Water corrosivity		
	Slight	Moderate	Severe
Hudson (1966)	0.015	-	0.61
Lamont (1981)	0.025	-	0.76

Considering the water quality (moderate to severe), pipes type material and the low maintenance services in Benghazi distribution system, growth rate "a" is set to 0.5 mm/year for all pipes. In addition, an initial roughness surface height e_0 of 0.2 mm was selected for all diameters encountered in the study system to nearly match the relative roughness value given in (Sharp & Walski., 1988) and to obtain a C factor of 140 for new ductile pipe. Initial values of pipe roughness coefficients for different pipe diameters and ages are tabulated in Table 3. However, pipe friction factors were calibrated and adjusted after the first run of the model to represent the effect of aging over the simulation duration. The adjustments were made to account for increase in resistance to flow caused by corrosion as a pipe ages and to simulate the real system behavior.

3-1 HYDRAULIC MODELLING (SIMULATION)

WDS modeling started by the end of 1960s, and the earliest models for water-distribution network design were developed for branched networks as reported by (Karmeli et al., 1968) and (Schaake et al.,1969). Applicability of these approaches to branched systems and assuming a linearity to simplify the hydraulic solutions were a major drawback facing water utilities. Thereafter, nonlinear models were designed to provide a better mathematical way to characterize the inherent nonlinearity in real dynamic systems and to improve modeling efficiency. Although, many nonlinear models were proposed [e.g., (Schaake et al., 1969) and (Liang 1971), they could not overcome the limited applicability of the branched systems and they did not have any computational advantages over the linear programming based approaches. Many primitive attempts toward modeling looped networks have been started and, probably, one of the most important efforts was presented by (Alperovits et al., 1977) who searched for some iterative methods to generate a sequence of data estimates, converging towards a local optimum of the criterion function. They exploited relative strength of the linear programming methodologies and used the fundamental linear programming formulation, and they approached this problem by using a gradient-search method and they proposed a linear programming gradient method (LPG) for optimization of a looped water distribution system. By completing feeding the model with the required inputs, model analysis (simulation) remains as the last step in the model building before introducing it for calibration. Model analysis is the final phase simulating the behavior of the real system. From which all the characteristics of hydraulic and water quality elements of the network can be obtained. Two types of model analyses which might be conducted on drinking WDSs: steady-state and extension period simulation EPS (Rosman 2000). Adoption a specific model simulation is dependent on objective of modeling (Fisher et al 1996). A steady-state simulation provides information regarding the equilibrium flows, pressures, and other variables defining the state of the network for a unique set of hydraulic demands and boundary conditions.

Table 3 Initial C- Factor (roughness coefficient) for different pipe diameters and age

	Diameter (mm)										
	1000	900	800	700	600	500	400	300	250	200	150
Age (year)	C – Factor										
5	113.55	111.85	109.95	107.79	105.30	102.35	98.75	94.10	91.16	87.55	82.90
10	102.96	101.26	99.36	97.20	94.71	91.77	88.16	83.51	80.57	76.96	72.32
15	96.62	94.92	93.02	90.86	88.37	85.42	81.82	77.17	74.23	70.62	65.97
20	92.08	90.38	88.48	86.32	83.83	80.88	77.28	72.63	69.68	66.08	61.43
25	88.54	86.84	84.93	82.78	80.29	77.34	73.74	69.09	66.14	62.54	57.89
30	85.64	83.93	82.03	79.87	77.38	74.44	70.83	66.18	63.24	59.63	54.99
35	83.18	81.47	79.57	77.41	74.92	71.98	68.37	63.72	60.78	57.17	52.53
40	81.04	79.34	77.44	75.28	72.79	69.84	66.24	61.59	58.64	55.04	50.39
45	79.16	77.45	75.55	73.39	70.90	67.96	64.35	59.70	56.76	53.15	48.51

In other words, steady-state simulation assumes all system components (e.g., pumps and tanks) and processes do not change with time. Though, real water distribution systems are seldom in a true steady state. However, the mathematical construct of a steady state is indispensible to analyze specific conditions such as peak demand times, fire protection usage, and system component failures in which the effects of time are not significant. There are many cases, however, for which assumptions of a steady-state simulation are not effective, or a simulation model is required that allows the system to change over time. For example, to monitor and study effects of changing water consumption over time, fill and drain processes of tanks and scheduling of pumps and valves. In such cases, an extended-period simulation (EPS) is needed. The EPS is created by given hydraulic time steps to steady-state simulation. After each steady-state step, the system boundary conditions are reevaluated and updated to reflect changes in the system behavior (junction demands, tank levels, pump operations, and so on). Then, another hydraulic time step is taken, and the process continues until the end of the simulation. An extended-period simulation can be run for any length of duration, depending on the objective of the model. The most common simulation duration is typically a multiple of 24 hours, because the most used category for demands and operations is a daily base. In some cases, however, simulation duration runs only a few hours into the future to predict immediate changes in tank level and system pressures. As mixing process in tanks is a major process in advective-reactive reactions occurs in water distribution systems. Moreover, mixing is the physical process stand behind the decay of disinfectant agent in water systems. This process, normally, takes long time to be stabilized. In addition to the capacity operation of storage tanks in the hydraulic system and the level variations between the hours of consumption, all that implied necessitates to longer simulation duration of several days in order to model the water quality. Generally, even in a system that have adequate storage capacity, simulation duration of 72 hours is considered useful in better determining the tank draining and filling characteristics. Selection of time step is an important decision when running EPS. The time step represents how the system changes gradually from one increment to next. Typically, one-hour time steps, found satisfy for water hydraulic modeling. Less time step, however, is needed for water quality modeling (minutes rather than hours). Generally, for highly variable processes, decreasing the time step can improve the accuracy of the simulation. Run simulation for different time step can also be used to select the proper increment needed. The selected times for hydraulic and water quality models developed for the case study are tabulated in Table 4. The simulation of Benghazi water distribution system using EPANET and the mentioned inputs is shown in Fig.5

Table 4 Times options of the developed hydraulic and water quality models

Property	Time (hours)
Hydraulic duration	72:00
Hydraulic time step	1:00
Quality duration	300:00
Quality time step	0:5

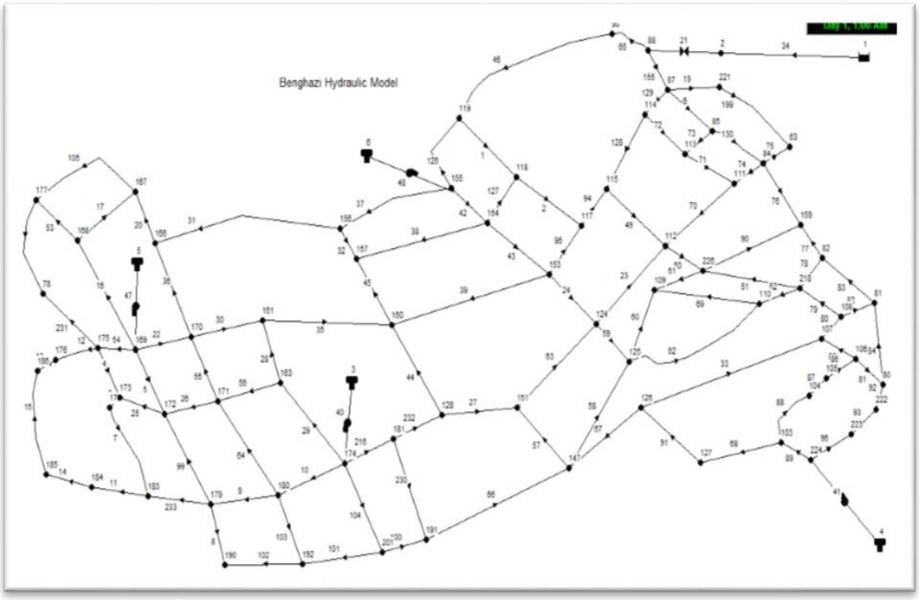


Fig. 5 Hydraulic model for Skeletonized Benghazi water distribution system (lengths are compressed so model can be seen in one screen).

Fig 5 represents the hydraulic model constructed by EPANET software, where lines represent pipes of the network, numbers on pipes are flow values, arrows are direction of flow ,pipes are connected by joints (nodes), one reservoir and 3 tanks and 3 pumps are presented as well in the model.

3-2 WATER QUALITY MODELING

The concept of modeling quality of water in WDS was first introduced in 1980 (Clark et al., 1986). The dynamic behavior of water networks incorporated water quality models were developed in 1986 (Grayman et al., 1988). The real improvement of those models were in the

1990s by the introduction of the public domain EPANET model (Rossman, 2000) and other Windows-based commercial WDS models. Mathematically, water quality models represent phenomena of reaction kinetics of disinfectant agent in bulk water phase and at pipe wall with a set of mathematical equations. These equations are solved under an appropriate set of boundary and initial conditions to predict the variation of quality of water throughout the distribution system (Walski et al., 2003). Water quality modeling is a direct extension of hydraulic network modeling. Therefore, water quality simulations use a network hydraulic solution not only as part of their computations, but also as a starting point in performing a water quality analysis. However, water quality models require for disinfectant matters, information for constituents decay or grow over time. Modeling fate of residual chlorine is one of the most common applications of network water quality models. Recalling, studies have shown that there are two separate reaction mechanisms for chlorine decay, one involving reactions within bulk fluid and other involving reactions with material on or released from pipe wall (Vasconcelos et al., 1997). Therefore, rate of reactions for both bulk and wall are required for water quality modeling. Moreover, water quality is a function of water chemistry and physical characteristics of a distribution system (e.g., pipe material and age), in addition to time. Residence time of water (water age) in a distribution system forms a critical factor influencing water quality. Water age is essential for evaluating the decay of disinfectant residual and the formation of disinfection by-products in distribution systems. A calibrated hydraulic model can be used to evaluate water age. The solution methods used in simulation of water age is actually specific applications of the method used in constituent analysis. Water entering a network from a source is considered to have an age of zero and the water age modeling considers the cumulative residence time for each parcel of water moving through the network.

3-2-1 Initial Water Quality values - A water quality model requires initial quality associated with external inflows to a system and water quality throughout the system can be estimated at a start of the simulation, this is usually represented by initial required concentration of chlorine continuously enters a network to maintain chlorine content in a system within the allowable range (0.2 to 0.5 mg/l). Initial water quality values can be estimated based on field data. Alternatively, EPANET provides facilities to judge for best estimates of initial conditions (by assuming global bulk decay and order of decay equation). Then the model is run for a sufficiently long period of time so that the initial conditions, especially in storage tanks, do not influence the water quality predictions in the distribution system (Walski et al., 2003). Determining the required initial concentration of chlorine that should be given to the system to maintain the chlorine concentration within the allowable limits throughout the whole system to maintain microbiological quality and minimize bio-film formation throughout the distribution system is not an easy task. For the existing condition of the study network the initial chlorine concentrations were hard to be estimated, probably because of the extreme variations in pipes conditions. Although, a large number of arbitrary initial chlorine concentrations were tried for long period of simulation (300 hours). Epanet tool requires input values for global bulk decay coefficient, and initial chlorine concentrations in the reservoirs. Besides the chlorine concentration, the quality of the water in the two reservoirs was identical, and it was not possible to set any other characteristics for the water. The input characteristics for the network tanks included only dimensional characteristics without possibility of choosing their material.

3-2-2 Mathematical formulae for chlorine demand:

Bulk decay is kind of reactions occurs within the water bulk and is a function of disinfectant concentrations, reaction rate and order, and concentrations of the formation products. A generalized expression for n^{th} order bulk fluid reactions is developed in Eq. 2 (Rossman, 2000).

$$\mathrm{Ct} = \pm \mathrm{k}\,\mathrm{C}^{\mathrm{n}} \tag{2}$$

Where; C_t is the concentration at time t along simulation time, (mass/m^3/unit time) ,k is the overall reaction rate constant (bulk + wall), C is concentration (mass unit/ m^3) and n = reaction rate order constant. The most commonly used reaction model for chlorine decay in water bulk, is the first order decay model, (DiGiano & Zhang 2005), However, concluded that a zero-order overall kinetic model is well suitable for describing the overall chlorine decay in a heavily tuberculated cast iron pipe, whereas, first order overall kinetic model is found suitable for a new cement-lined ductile iron pipe. Moreover, (Vasconcelos et al., 1997) found that chlorine decay in distribution systems can be characterized as a combination of first-order reactions in the bulk liquid and first-order or zero-order mass transfer-limited reactions at the pipe wall. Therefore, chlorine decay is often simplified to first-order kinetics. A first order decay is equivalent to an exponential decay, represented by Eq. 3

$$C_t = C_o e^{-kt} \qquad (3)$$

Where; C_t = concentration at time t (mass unit/ m^3), C_O = initial concentration (at time zero), k = reaction rate (1/time unit). For first order reactions, units of k are (1/T) with values generally expressed in 1/ days or 1/hours. Eq. 3 states that, chlorine concentration “C_t” in mg/l can be found at any time “t” in days provided knowing initial chlorine concentration “C_O” in mg/l and overall decay rate of chlorine “k” in 1/day. In this paper only bulk decay is considered (for reasons already mentioned earlier), therefore by finding chlorine decay rate (K_b), initial concentration (required dose) can be found.

3-2-3 Bulk decay rate inputs

First-order rate constants for chlorine decay in the bulk flow can be estimated by performing a bottle test in the laboratory. Water samples are stored in several amber bottles and kept at a constant temperature at several periods of time, a bottle is selected and analyzed for free chlorine. At the end of the test, the natural logarithms of the measured chlorine values are plotted against time. The rate constant is the slope of the straight line through these points. There is currently no similar direct test to estimate wall-reaction rate constants. Instead, one must rely on calibration against measured field data. Accordingly, 75 samples of water were collected from different points of Benghazi network and analyzed. The test carried out by Engineering consultant office for municipality of Benghazi within their study titled "Project of assessment of existing status of Benghazi water distribution system, contract no.179/1370 (2003)", owner Municipality of Benghazi. The bulk chlorine decay coefficient was determined experimentally under laboratory conditions from the data collected from water supplied to the system. Bulk decay is measured by recording the chlorine concentration, at time intervals, from glass bottles. All tests were conducted at constant room temperature, using the same water and a similar level of mixing, which is almost completely mixed, and all samples having the same initial chlorine concentrations, thus ensuring that all other conditions not under study were consistent. The test carried on the samples every one hour (t) for 24 hours and with initial concentration starts with 0 mg/l till 10 mg/l (one step increment) to see how chlorination is decay in water bulk. The results of the tests is shown in Fig 6 from which K_b can be found for any C_0. Average of 1800 test readings for each hour is given in table 5.

Table 5, Lab. tests results average of residual chlorination concentration for Benghazi water network.

t(hr)	Ct (mg/l)	t(hr)	Ct (mg/l)	t(hr)	Ct (mg/l)	t(hr)	Ct (mg/l)
1	0.43	7	0.42	13	0.27	19	0.18
2	0.53	8	0.38	14	0.257	20	0.15
3	0.54	9	0.34	15	0.237	21	0.16
4	0.51	10	0.31	16	0.22	22	0.15
5	0.48	11	0.30	17	0.20	23	0.14
6	0.45	12	0.28	18	0.19	24	0.13

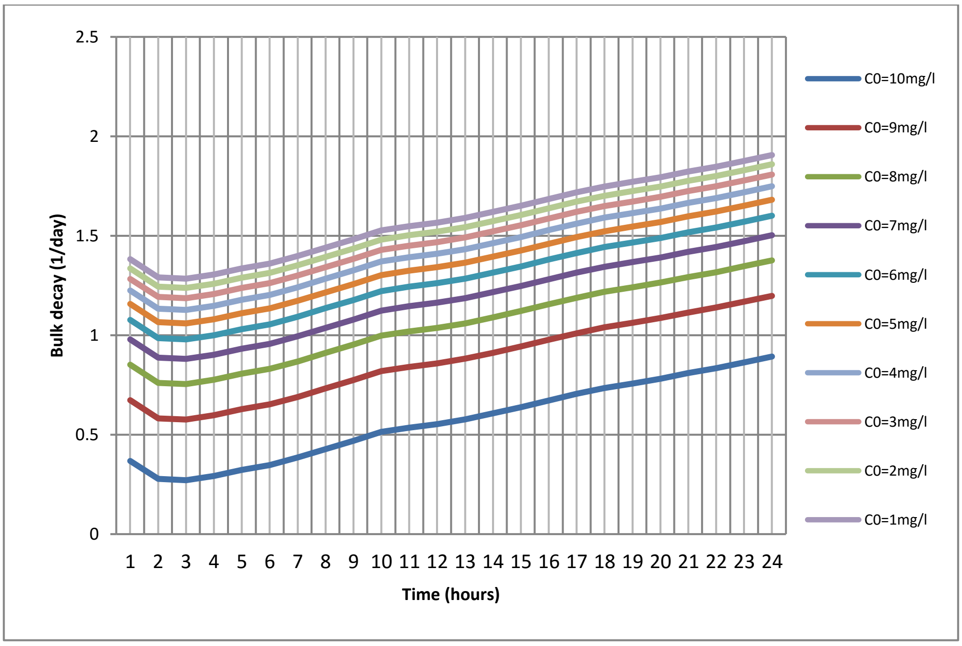


Fig 6 bulk decay for different assumed initial concentration.

From fig 6, the relationship between C_0 and K_b have a general model of first degree (straight line). Accordingly the chlorine bulk decay can be measured for any initial chlorine concentration by Eq. 4.

$K_b = a\,t + b$ (R2=0.976) (4)

Where; K_b is bulk decay at any time t (temperature = 20 C, Total Organic Carbonation TOC ranges from 1.5 to 3 mg/l, and pH ranges from 6.5 to 8.5). Table 6 represents a and b parameters for different C_O.

Table 6 Parameters of K_b for different C0

Initial chlorine concentration C0 (mg/l)										
Parameter (eq 4)	10	9	8	7	6	5	4	3	2	1
a	0.02	0.0.3	0.03	0.028	0.028	0.028	0.028	0.03	0.03	0.03
b	0.22	0.52	0.700	0.82	0.92	1.00	1.07	1.13	1.22	1.23

Note: a and b are parameters given in equation 4, from which the chlorine bulk decay can be measured for any initial chlorine concentration

Table 7 comparison between concentration of chlorine in Lab Ct L and in hydraulic simulation Ct S

t(hr)	Ct L (mg/l)	Ct S (mg/l)	t(hr)	Ct L(mg/l)	Ct S (mg/l)	t(hr)	Ct L (mg/l)	Ct S (mg/l)	t(hr)	Ct L (mg/l)	Ct S (mg/l)
1	0.43	0.33	7	0.42	0.38	13	0.27	0.19	19	0.18	0.12
2	0.53	0.39	8	0.38	0.37	14	0.26	0.18	20	0.15	0.10
3	0.54	0.39	9	0.34	0.36	15	0.24	0.18	21	0.16	0.09
4	0.51	0.42	10	0.31	0.28	16	0.22	0.16	22	0.15	0.09
5	0.48	0.43	11	0.30	0.22	17	0.20	0.15	23	0.14	0.08
6	0.45	0.38	12	0.28	0.19	18	0.19	0.14	24	0.13	0.07

4. RESULTS AND DISCUSION

Using eq. 2, bulk reaction rate average (k_b) for 24 hours was found, and represented in Fig 7. A high correlation of $R^2 = 0.9763$ was noted from the curve fit equation , then reaction rate " k_b" can be found at any time "t". It is clear that the reaction rate increases in the first hour then from the second hour starts to decrease to reach its lowest value at the end of the test (24hrs) with rate equal to -0.048 1/day. Minus (negative) sign is a result of ($\ln C_t / \ln C_o$) as shown in Fig 7.

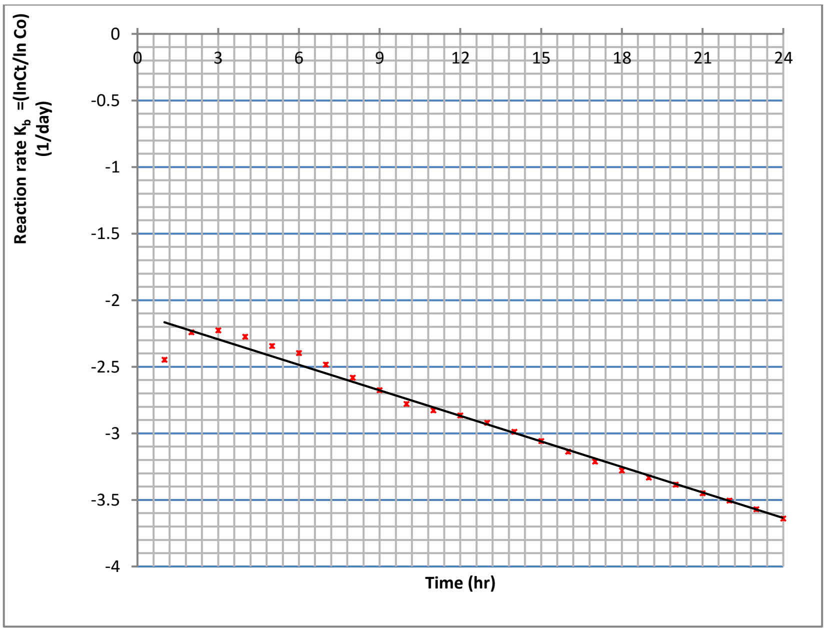


Fig 7 reaction rate of bulk decay "k_b" for study water network found in Lab.

4-1 CALIBRATION:

As mentioned earlier, Epanet tool requires input values for global bulk decay coefficient, and initial chlorine concentrations in the reservoirs, therefore simulations with 0.048 1/day (as found in Lab) was given as global bulk decay, and 10 simulations trails were run for the ten assumed values of C_0. Despite a large number of arbitrary initial chlorine concentrations were tried (from 0 to 10 mg/l in hydraulic model for long period of simulation (300 hours), however only last 24 hours in simulation were considered for calibration purpose. The amount of residual chlorine concentration C_t which is close to the amount stipulated by (WHO 2008) (0.2 to 0.5mg/l) are found at C_0 equal to 4 mg/l. This mount or dose can be pumped to the water distribution system of Benghazi to maintain the water in network safe and healthy. Notifying K_b found from the simulation is equal -0.055 which is slightly larger than of K_b found in Lab, Fig 8 . This difference between kb based Lab result and Kb based simulation is perhaps due to considering pipe age in simulation in addition to difficulty to represent all the conditions of the Lab. However comparison between C_t found in Lab and C_t found from simulation insured the calibration between them as given in Table 7. Statistically, the average, standard deviation and correlation between C_t (Lab) and C_t (Simulation) insure the significance of K_b value obtained and thus the chlorination initial dose C_0, Table 8.

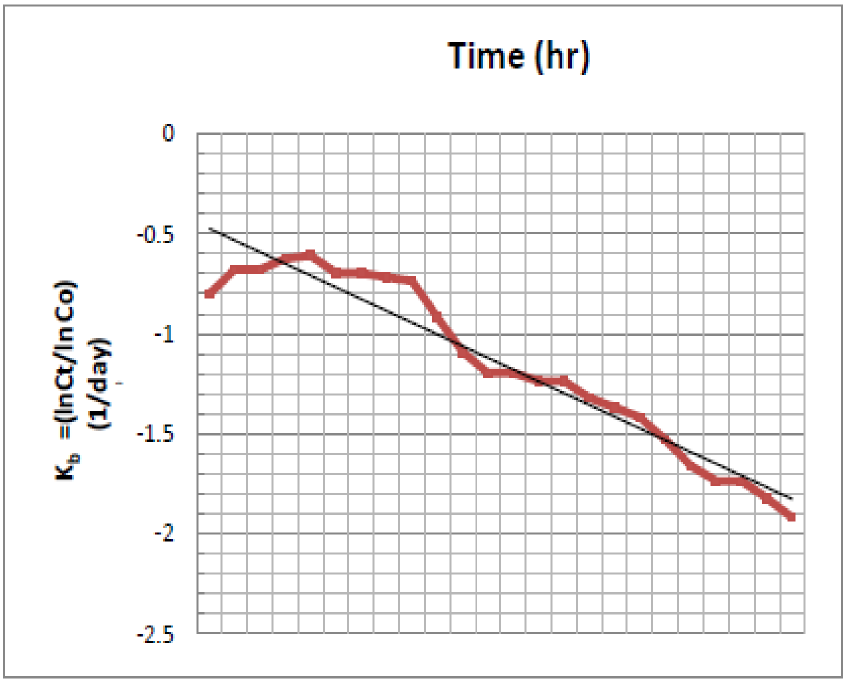


Fig 8 shows K_b rate found from hydraulic model simulation

Table 8 main statistical characteristics of the C_t found in Lab and in hydraulic simulation

Statistical significance	C_t S	C_t L	K_b	
Average	0.22	0.30	From Lab	From model
Standard Deviation	0.13	0.14	-0.048	-0.055
Correlation	0.96		0.87	

5. CONCLUSION:

Chlorine is one of the most important chemical agents used to disinfect drinking water. This chlorine interacts with the bulk water as it interacts with the wall pipes, causing its decay. Thus the value given at the source will decay as a result of its interaction with the elements in the water or the walls of the internal pipes, so it is important to know the residual value of chlorine, which is must be in the range 0.2 to 0.5 mg/l all the time in all the network, otherwise the water will not be suitable for drinking. Thus, it is necessary to determine the coefficient of chlorine decay in the network so that we can determine the optimal initial concentration of chlorine that must be pumped into the network. This paper aimed to determine the initial concentration of chlorine that must be pumped into the Benghazi water network in order to maintain its quality. To achieve this goal, a hydraulic model of the network was constructed using the EPANT program. Where all elements of the network were represented, including the age of the pipes. The simulation was operated for 300 hours using the value of the bulk decay rate that was determined in the laboratory (-0.0481/day) by the Engineering Consulting Office of the Municipality of Benghazi. Accordingly the optimal initial concentration of chlorine was determined (4 mg/l), which gives residual values of chlorine close to the values obtained from the laboratory, and by plotting the logarithm of these concentrations with the logarithm of the optimal initial concentration, the decay rate was found equal to -0.055 1/day.

REFERENCES:

- Alperovits, E., & Shamir U. 1977. Design of optimal water distribution systems. Journal of Water resources research 13(6): 885-900.
- Brown, D., Bridgeman, J. & West, J.R. 2011. Predicting chlorine decay and THM formation in water supply systems. Reviews in Environmental Science Biotechnol 10:79–99
- Clark, R.M., Grayman, W.M., Males, R. M., & Coyle, J.A. 1986. Predicting Water Quality in Distribution Systems. Proceedings of the American Water Works Association (AWWA). Distribution System Symposium. American Water Works Association. Denver, Colorado.
- Clark, R. 1998. Chlorine demand and TTHM formation kinetics: a second order model. Journal of Environmental Engineering, ASCE 124 (1): 16-23.
- Clark R. M and Mano Sivaganesan Journal of Water Resources Planning and Management/ Volume 128 Issue 2 - March 2002
- DiGiano, F.A., Zhang, W. & Travaglia, A. 2005. Development of the mean residence time from tracer studies in distribution systems. Journal of Water Supply: Research and Technology - Aqua 54(1):1-14.
- Engineering consultant office for municipality of Benghazi, "project of assessment of existing status of Benghazi water distribution system, contract no.179/1370 (2003)", owner Municipality of Benghazi.

- Fisher, I., Kastl, G., 1996. Numerical modeling of water quality in distribution systems. In: Proceedings, Australian Water and Wastewater Association WaterTECH Conference, May 1996,Sydney, Australia, p. 461ff.
- 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):439-452
- Hudson, W.D. 1966. Studies of Distribution System Capacity in Seven Cities. Journal of the American Water Works Association 58(2):157.
- Karmeli, D., Gadish, Y. & Meyers, S. 1968. Design of optimal water distribution networks. Journal of Pipeline Divison, ASCE 94(1): 1-9.
- Lamont, P.A. 1981.Common Pipe Flow Formulas Compared with the Theory of Roughness. Journal of the AWWA 73(5):274-283.
- Liang, T. 1971. Design of conduit system by dynamic programming. Journal of the Hydrulic Division, ASCE 97(3): 383-393.
- Menaia, J. Coelho S.T., Lopes A., Fonte, E. & Palma, J. 2002. Dependency of Bulk Chlorine Decay Rates on Flow Velocity in Water Distribution Networks. World Water Congress, Melbourne, Australia
- N.B. Hallam, John R.West,C.F.Forester, J.C.Powell, and I. Spancer .The Decay of Chlorine Associated with the Pipe Wall in Water Distribution System. Water research, vol.36, pp.3479-3488, 2002
- Rossman, L. 2000. EPANET 2 USERS MANUAL. U.S.
    - Environmental Protection Agency Cincinnati.
- Sarbatly, R. HJ. & Krishnaiah, D. 2007. Free chlorine
    - residual content within the drinking water distribution system. International Journal of Physical Sciences 2(8):196-201
- Sharp, W.W., & Walski, T. M. 1988. Predicting Internal Roughness in Water Mains, Journal of AWWA 80(11): 34-40.
- Schaake, J. & Lai, D. 1969. Linear programming and dynamic programming—application of water distribution network design. Report 116, MIT Press, Cambridge, Mass.

- Silja, T., Helena, R. & Didia, C. 2008. Water Supply System Performance for Different Pipe Materials Part I: Water Quality Analysis. Journal of Water Resources Management 22(11):1579–1607.
- Vasconcelos, J.J., Rossman, L.A., Grayman, W.M., Boulos, P.F., & Clark, R.M. 1997. Kinetics of Chlorine Decay. Journal of the American Water Works Association 89(7); 54.
- Walski T M., Chase, D.V., Savic, D.A., Grayman, W., Beckwith, S., & Koelle, E. 2003. Advanced water distribution modeling and management, First Edition, HAESTAD PRESS Waterbury, CT USA.
- Guidelines for drinking-water quality - 4th ed. 2008.