Hassan Al-MOUSAWEY✉

Basim Sh. ABED

University of Baghdad, Collage of Engineering, Baghdad, Iraq

SIMULATION AND ASSESSMENT OF WATER SUPPLY NETWORK FOR AL-NASIR NETWORK AT AL-NAJAF GOVERNORATE

Introduction

Water is one of the basic necessities for healthy existence. Having a sufficient supply of drinkable water promotes both the welfare of the population and the best possible growth of many industries (Albadry, 2017; Ho, Puika & Kasih, 2020). The inefficient everyday functioning of a water distribution system (WDS) imposes considerable consequences even if failure occurrences are unavoidable and frequently spectacular and expensive. Performance measurement is essential in engineering the operation and management of any WDS (Hussein, 2021). Modelling WDS is used to acquire an accurate idea of the network's operation and to identify the causes and situations that influence the network's performance (van Summeren & Blokker, 2017).

Managing the amount of free residual chlorine in water distribution systems is crucial for ensuring that end-user tap water is safe for consumption. Residual chlorine level should be between 0.2 and 0.5 $mg \cdot L^{-1}$ to eliminate pathogens without causing bad taste (Alsaydalani, 2019; Casas-Monroy et al., 2019). There is a variety of software for modelling distribution networks, with EPANET, Loop, and WaterCAD being the most prominent. But WaterGEMS has the most capabilities of all, and it was employed in this research (Kadhim, Abdulrazzaq & Mohammed, 2021).

Abdulsamad and Abdulrazzaq (2022) created a hydraulic network model using

✉hassan.almousawey2010m@coeng.uobaghdad.edu.iq

ArcMap 10.8 and WaterGEMS. The network's source nodes are measured with a handheld electronic ultrasonic flowmeter. Using a pressure gauge at eight junctions, the system was calibrated. The network's pressure varied from 8 to 21 m, while the main pipes' velocity was 0.5–2 m·s^{-1}.

Patel, Sahoo and Mohanty (2020) used WaterCAD software to analyse the water distribution network in the municipality of Dilla town. Through simulation, fixed pressure reduction valves, also known as PRVs, were placed in the areas where they would be most efficient in lowering both the pressure and the amount of water lost to leakage. Initially, the existing system had an average water distribution pressure of 58 m when it was first built. The network was optimised, and as a result, the average pressure decreased to 44 m; therefore, the amount of leakage decreased by an average of 24%. According to the findings of the study, the implementation of PRVs in the water distribution system is a potentially useful method for minimising water loss due to leakage since it lowers the pressure in the water distribution networks.

Rai and Lingayat (2019) used EPANET software, the Hardy Cross methodology, and the Newton–Raphson method to analyse and distribute a water distribution system appropriately. Hydraulic variables such as pressure and flow were simulated. All flows and velocities were validated to be able to supply the network with an acceptable amount of water. By computing the residual head at each node using elevation as an input, researchers were able to deduce other significant flow parameters, including nodal demand, velocity, and residual head. The researchers discovered that complex networks could be addressed fast. Head loss decreases as the number of iterations increases, and the acquired values are checked by balancing flows at each point.

Hussien Al-Mansori, Mizhir Al-Fatlawi and Al-Zubaidi (2020) conducted a research in Iraq on Babylon University's pipe network to determine residual chlorine levels in drinkable water. This effort includes determining the number, position, and dose of needed onsite chlorine injection points in an effort to increase chlorine concentrations in the networks to acceptable levels. Between December 2016 and April 2017, 10 sample locations were chosen from Babylon University's water network. The concentration of residual chlorine was measured by taking samples three times a day, twice a week. The findings were evaluated in light of WHO criteria (World Health Organization [WHO], 2022). It was determined that the bulk decay coefficient is –1.18 daily while the wall decay coefficients range from –0.01 to –0.91 daily. The main conclusion of the study is that re-chlorination stations can be used to increase chlorine levels in the pipe network.

There is no previous research on the hydraulic performance and water quality of the water distribution network in Najaf city. The primary objective of this research is to evaluate the hydraulic capacities of Al--Nasir water distribution network in Al-Najaf Governorate through field measurements and the use of the WaterGEMS software. In addition, it intends to investigate and assess the chlorine deterioration in the distribution network.

Materials and methods

Study area

Abu-Talib, Al-Nasir, Al-Mohandisin, Old Milad, and Al-Fao Districts are located in the northern section of Al-Najaf Governorate, as shown in Figure 1. All of these districts are supplied by a single network. They have a combined surface area of 5.57 km^2, a population of 91,000 capita. According to the 2021 local estimation of population, drinking water in Al-Najaf city is treated by the main water treatment project of Al-Najaf city. Then water is pumped to the main pump station located approximately at the centre of the city, from there, seven mains distribute the water to the city, the network of the study area is supplied by a single main

pipe of a 500 mm diameter, and two pumps pumping water to the specified network working alternatively for eight-hour period. All the pipelines in the network are tested for 10 bar working pressure.

For the current analysis, a water distribution network model was constructed that included main and secondary pipes, neglecting network laterals. The total pipe length of the network is 68.2 km.

Field measurements and observation data

The data and designs for the network infrastructure were received from the municipal authorities of the water supply, while additional data on the pump servicing the network and operating hours were gathered from

FIGURE 1. Area of study within Al-Najaf Governorate

FIGURE 2. The locations of fieldwork measurements

Najaf city's main pump station. To correctly generate the model utilising WaterGEMS––ArcGIS interface, GIS plans were obtained from the water department. At the end-user taps in 10 locations distributed along the network (shown in Fig. 2), pressure readings were performed using a glycerine-filled pressure gauge preceded by an air venting valve (shown in Fig. 3) connected directly to the first connection inside the house.

The measurements were performed during peak demand hours. For the assessment of residual chlorine, water samples were taken from the network using clean sampling bottles.

FIGURE 3. The apparatus used for the pressure head measurements

Determination of residual chlorine coefficients

The decrease of residual chlorine in the water distribution system occurs in both the bulk water and the pipe walls. The wall decay is a surface area process, whereas the bulk decay is a volume-based degradation process.

Bulk decay coefficient (K_b). Pipe characteristics have little impact on the bulk flow response coefficient. The chemical makeup

of water determines its qualities. To establish its value, a laboratory test was carried out (Ozdemir & Ucak, 2002; Mostafa, Matta & Halim, 2013). The laboratory tests were performed by collecting eight water samples from various locations, then, each water sample was split into 13 parts, and the Lovibond Comparator 2000+ device was used to measure residual chlorine concentration.

At time zero, the first sample was measured. Then, the remaining samples were tested at one-hour intervals, and the findings were plotted against time, as seen in Figure 4. This method was performed on all eight water samples to achieve the most consistent results.

The first-order decay model assumes that the chlorine concentration will decline exponentially and is the most widely used one (Hua, West, Barker & Forster, 1999; Castro & Neves, 2003).

According to the equation $C = C_0 e^{-kt}$, where C is the chlorine concentration at the time (t), C_0 is an initial chlorine concentration, k is the decay rate in min^{-1}, t is the time in min. The calculated bulk decay coefficient was taken to be equal to -0.095 h^{-1}, which has the highest regression rate ($R^2 = 0.9867$) and corresponds to -2.28 day^{-1}. The negative sign refers to the reduction of residual chlorine with time.

Wall decay coefficient (K_w). The coefficient of wall deterioration is governed by the condition of the pipe wall and the lining material. It is affected by the interaction between the bulk flow and the wall contact. The mass transfer coefficient, which is based on the molecular diffusivity of the measured material, influences the bulk pipe flow rate. Amount of $1.44 \cdot 10^{-9}$ $m^2 \cdot s^{-1}$ is the chlorine diffusivity at a water temperature of 25°C. The network comprises PVC--HDPE and ductile iron pipes; however, because the network is less than 10 years old, the K_w value for the PVC pipes is assumed to be zero. The expected K_w value for ductile iron pipes is equal to -4 $mg \cdot m^{-2} \cdot day^{-1}$ (Nagatani et al., 2008). Then the value of K_w was adjusted until an acceptable level of fitness was achieved.
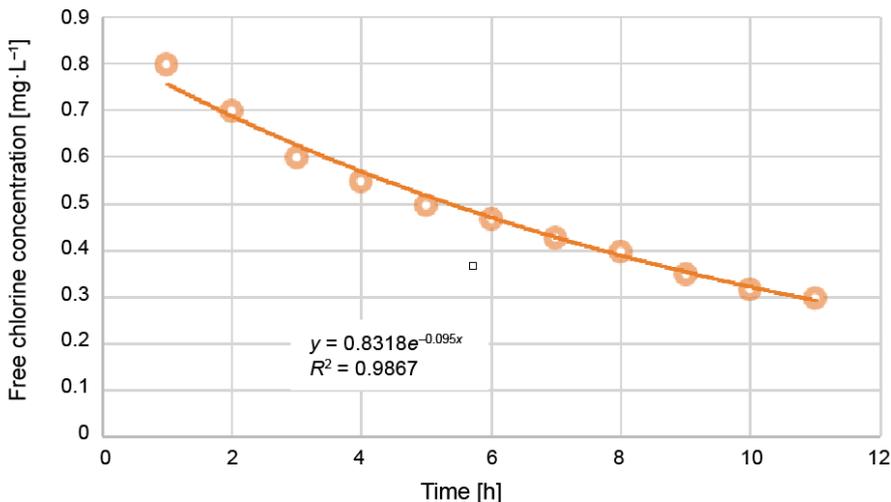
FIGURE 4. First-order adjustment of the testing results

Bentley WaterGEMS

Windows application WaterCAD was designed by Haestad Methods Inc. of Cincinnati, Ohio. WaterGEMS is the most recent version of the WaterCAD software, and it is a straightforward program for simulating or designing water distribution networks. WaterGEMS offers a thorough yet user-friendly decision-support tool for water distribution networks. The software helps you gain a better understanding of how infrastructure functions as a system, how it responds to operational scenarios, and how it should expand as population and demand rise. Engineers and utilities may use WaterGEMS to evaluate, build, and improve water distribution systems in the areas of discharge, pressure head, constituent concentration analysis, and pump simulation in different scenarios, with the condition that accurate network data is available to create an accurate model that truly represents the network under study (Haestad Methods Inc., 2003). Figure 5 illustrates the flowchart and required input data for WaterGEMS software.

In the current investigation, the Hazen––Williams head loss formula is utilised to calculate the amount of hydraulic head loss produced by friction with pipe walls. The reservoir's features, pipelines, connections, valves, and pumps were correctly modelled, and the system's demand was determined based on 350 L per capita daily. The demand of each distribution network junction was then determined based on the region covered by the individual junction. Pipe features were entered along with pump characteristics and valve properties. The network pipes are comprised of cement-lined ductile iron pipes with 500 and 300 mm diameters and PVC-HDPE pipes with 225 and 160 mm diameters. Since
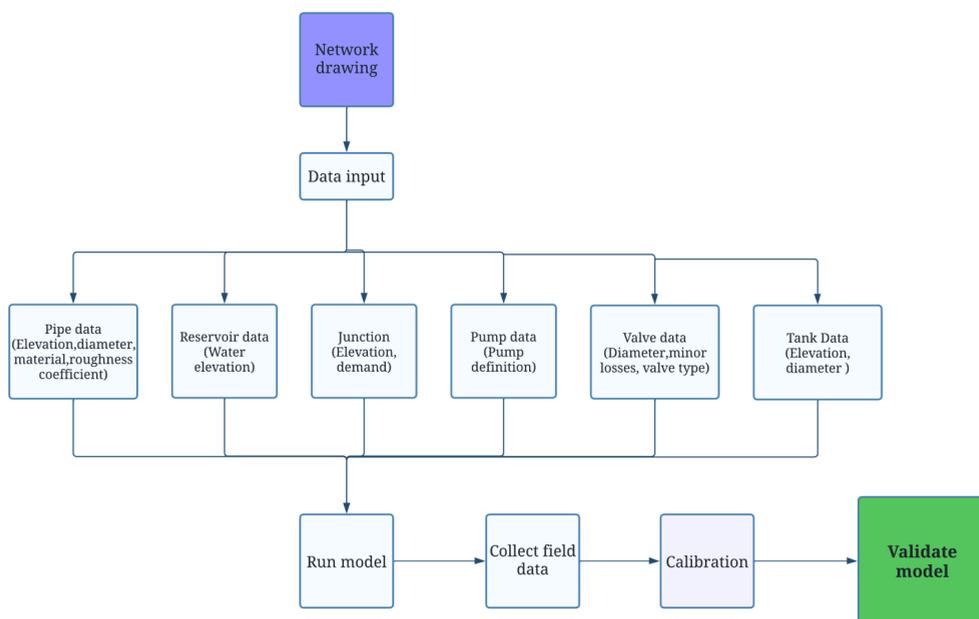
FIGURE 5. WaterGEMS software flowchart

the network is fairly new (less than 10 years old), the typical roughness coefficient (*C*) for the two types of pipes was assumed to be 120 and 150, respectively (Chin, Mazumdar & Roy, 2000; Jalal, 2008).

Model calibration

The calibration of a hydraulic model is to ensure the model's dependability. This phase in the modelling process entails fine-tuning a model until it matches field data over a specified period so that it may be used to predict system performance and evaluate alternative approaches (Sun & Sun, 2015; Hessling, 2017; Farhan & Abed, 2021). This process involves slight adjustments to the input values to generate output values that accurately represent the system (Khudair, 2015; Parady, Ory & Walker, 2021). Throughout the water distribution network, pressure head measure-ments are monitored and utilised to calibrate the pressure head levels. Multiple variables might affect pressure values. As a consequence, calibration may be accomplished by just adjusting the internal pipe rough-ness values or nodal demand predictions un-til adequate fitness is obtained between the observed and simulated pressure heads and flow values. This idea is based on the fact that pipes (*C*) values and system demands are often estimated, allowing space for error, in contrast to pipe lengths and diameters, which are measured directly. The calibration pro-cedure was performed between 4 and 5 AM during peak demand hours by slightly adjust-ing total system demand. Three sets of meas-urements were performed on three different days during peak demand hours. Results of measurements varied slightly. Thus, the data set having the highest *RMSE* value was se-lected to represent the calibration procedure.
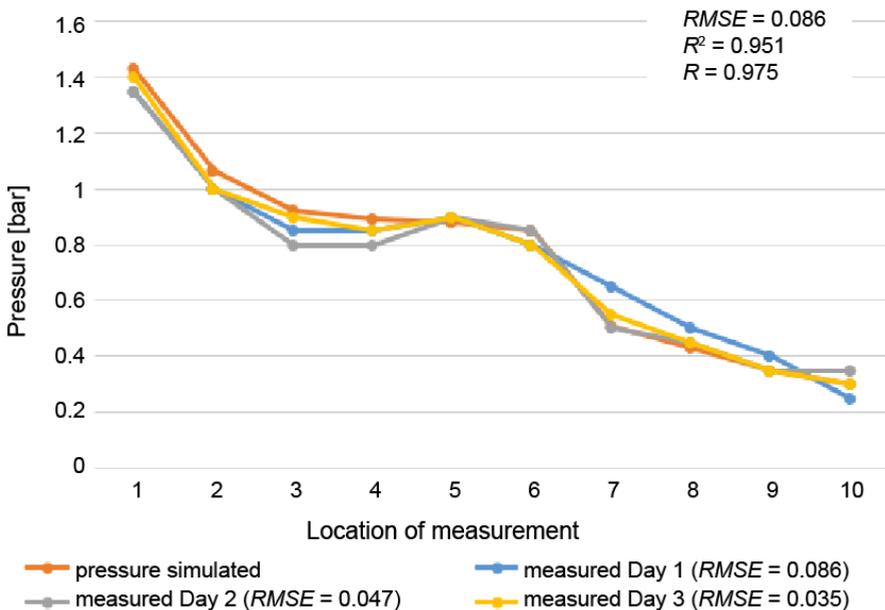
FIGURE 6. Comparison between simulated and measured pressure heads along different junctions

When comparing the observed and simulated pressure head values, the root mean squared error (*RMSE*), *R*-squared correlation (R^2) and the Pearson correlation coefficient (*R*) were used to demonstrate how close the two data sets were. Figure 6 compares measured and simulated pressure values with low *RMSE* values of 0.086 and high *R* and R^2 values of 0.975 and 0.951, respectively, indicating good agreement between simulated and observed values. On the other hand, the model was calibrated to provide an approximation of the wall decay coefficient (K_w), which was assumed to be –4 mg·m^{-2}·day^{-1} (as was mentioned earlier), value –2.3 mg·m^{-2}·day^{-1} was determined by trial and error to be the value for K_w that yielded the most plausible results in simulating the actual measurements in the network. For simulating residual chlorine concentrations, the model was calibrated similarly based on the results of concentration tests on 10 samples collected from various places within the network for three days.

Figure 7 illustrates the comparison between computed and field measured chlorine samples at maximum consumption hours with low *RMSE* value of 0.057 and high *R* and R^2 values of 0.917 and 0.84, respectively which shows high agreement between observed and simulated values.

Model verification

Similarly to the calibration process, three sets of field measurements on three different days were carried out to gather pressure head readings at various points throughout the network during the hours of moderate water usage between 10 and 11 AM. Flow-rate measurements were made at the location of the pump station and some other locations using an ultrasound flowmeter to estimate system demand. This was done to ensure the accuracy of the simulation of the modelled distribution network. Additionally, residual chlorine concentrations were measured at
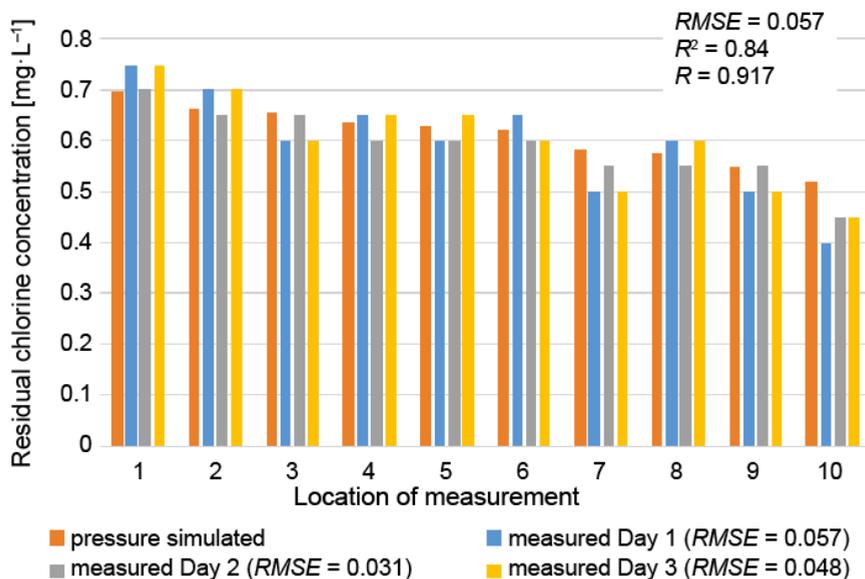
FIGURE 7. Comparison between simulated and field-measured residual chlorine concentrations

field measurement sites. The measurement set with the highest *RMSE* value was selected to represent the verification process. Table 1 compares the field observed to simulated pressure values, and the verification process' findings suggest that there is strong agreement between these measurements because of the negligible difference and low value of *RMSE* of 0.095 and high R and R^2 values of 0.985 and 0.97, respectively.

Table 2 compares the observed and simulated residual chlorine concentrations. The results indicate that the difference between the observed and simulated residual chlorine concentrations is low, and that the simulation achieves close results to the observed concentrations due to the small *RMSE* value of 0.05 and good R and R^2 values of 0.819 and 0.671, respectively.

TABLE 1. Comparison between calculated and measured pressure heads at various nodes

Node ID	Simulated pressure [bar]	Measured pressure [bar]	*RMSE*	R^2	R
J-1884	1.45	1.5			
J-1900	1.446	1.5			
J-1910	1.3	1.4			
J-1888	1.284	1.25			
J-1886	1.281	1.2			
J-1908	1.254	1.25	0.095	0.97	0.985
J-1906	1.112	1			
J-1904	1.066	1			
J-1894	0.987	0.85			
J-1896	0.902	0.75			

TABLE 2. Comparison between simulated and observed residual chlorine concentrations

Node ID	Simulated residual chlorine [mg·L^{-1}]	Measured residual chlorine [mg·L^{-1}]	*RMSE*	R^2	R
J-1894	0.686	0.7			
J-1886	0.644	0.65			
J-1896	0.632	0.65			
J-1888	0.624	0.5			
J-1884	0.589	0.6			
J-1906	0.578	0.55	0.05	0.671	0.819
J-1908	0.536	0.55			
J-1904	0.519	0.55			
J-1910	0.494	0.5			
J-1900	0.484	0.4			

Consequently, the hydraulic and residual chlorine simulation data may be relied upon when analysing the water distribution system.

Results and discussion

Pressure heads analysis

Using WaterGEMS software, a pressure heads analysis of the distribution network was carried out under steady-state circumstances during peak water demand hours, moderate water demand hours, and low water consumption hours. It was discovered that during the peak water demand hours, one pump was operating over its intended operating capability, mostly because of increased water use and continuous running of domestic pumps inside homes. Most network pressure heads were below acceptable levels, particularly at the centre of the network, and their values ranged from 0.2 to 1.3 bar; however, some junctions in the network had acceptable values of water pressure of 1.3––2 bar. This decrease in the pumping head is caused by the pump reaching its maximum discharge point due to increased demand. The simulation results show that the pressure heads for hours of moderate water demand are between 0.9 and 1.6 bar. For the third flow condition, which represents low water demand, the pressure heads reached the designed level because there was less water use and the domestic household tanks were full at this time. The pressure values were between 1.42 and 2.7. From field observation, most homes have their own pumps, which causes the water pressure in the network to drop, especially as the distance from the pumping station goes up. In Figures 8 and 9, a contour
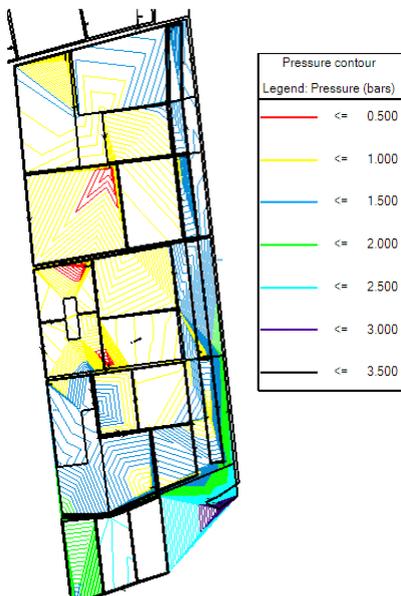
FIGURE 8. Contour map of pressure heads during maximum water demand

FIGURE 9. Contour map of the pressure heads during low water demand

profile shows the different pressure ranges in the network for different flow conditions. A possible solution for the present network's low pressures is to replace the existing pump with a newer model (locally obtainable) with a capacity of 3,000 m³·h⁻¹ instead of the current one (1,200 m³·h⁻¹). This upgraded pump will be able to supply the appropriate pressure at the network's near-end connectors. Figure 10 depicts the simulation result of the suggested method, with the lowest pressure value of 1.6 bar at maximum demand.

Velocity analysis

The simulation analysis shows that during maximum water demand, the flow velocities ranged between 0.5 and 2.59 $m \cdot s^{-1}$ for ductile iron pipes of 500 and 300 mm. The velocities in the PVC-HDPE pipes of 225 mm ranged between 0.1 and 2.65 $m \cdot s^{-1}$;

the drop of velocity in some of the 225 mm pipes is due to unrequired extra piping in some locations. The velocity values for 160 mm PVC-HDPE pipes ranged between

FIGURE 11. The ranges of velocities in the water distribution system at maximum water demand

FIGURE 10. Pressure heads with improved pump at maximum demand
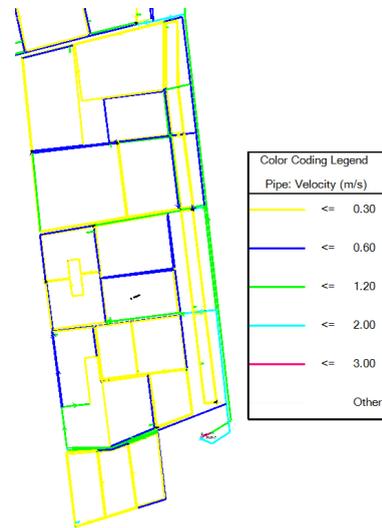
FIGURE 12. The ranges of velocities in the water distribution system at minimum water demand

0.1 and 1 m·s^{-1}. These values of flow velocity revealed that the velocity in the ductile pipes is mainly within the allowable limits for all times, while the velocity in the lateral pipes exceeds the allowable limit in the low demand periods and decreases to be below the permissible limit in the dead flow junctions. Figures 11 and 12 show the velocity distribution for the distribution system for high and low water demand periods.

Residual chlorine analysis

During maximum demand period, the values of residual chlorine in the network were in the range of 0.5–0.8 mg·L^{-1}, decreasing as the water reaches the middle parts of the distribution network. These ranges of residual chlorine are considered acceptable by local and international standards in terms of low limits but exceed the upper limit of 0.5 mg·L^{-1}. The contour profile in Figure 13 shows the concentrations of residual chlorine throughout the network. During low demand period, the values of residual chlo-

rine concentrations ranged between 0.45 and 0.76 mg·L^{-1}, which exceeded the upper limit according to international standards (0.2––0.5 ppm) (WHO, 2022). Figure 14 shows the contour profile of residual chlorine concentrations during low demand period.

Conclusions

The network simulation using WaterGEMS software proved successful, with *RMSE* values ranging between 0.08 and 0.1, and 0.05 and 0.06 for the two simulations, respectively. It was revealed that there is a deficit in water pressure values nearly at the centre of the WDS, where the pressure heads drop below 1 bar due to the current one pump operation, particularly during peak demand hours; however, when there was less demand, the network performed adequately, and water pressure values were between 1.42 and 2.7 bar. Velocity in the main pipes of the distribution network is acceptable. However, velocity might increase or decrease in lateral
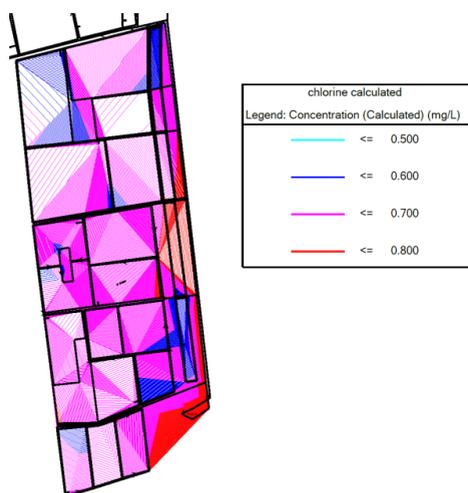
FIGURE 13. Values of residual chlorine during maximum demand period
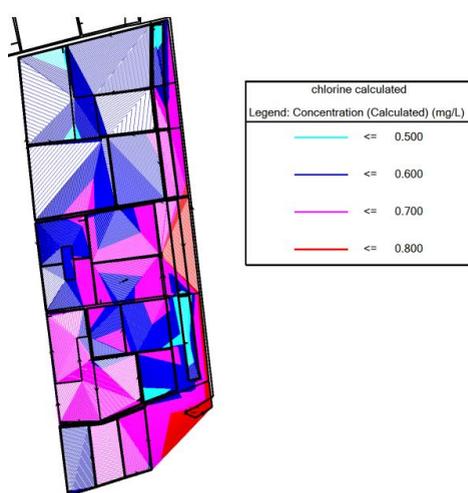
FIGURE 14. Values of residual chlorine during low demand period

pipes because of factors such as low demand or dead junctions. The pump that supplies the distribution network has inadequate specifications and cannot run within its optimal range. The simulation also showed that the residual chlorine levels were above the lower permissible limits at most locations throughout both testing periods, between 0.45 and 0.8 ppm.

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Summary

Simulation and assessment of water supply network for Al-Nasir network at Al-Najaf Governorate. This study simulates and assesses the hydraulic features and residual chlorine in Al-Najaf city's water supply network using WaterGEMS. Field and laboratory work was done to determine pressure heads, velocities, and chlorine residual. Constructed model was validated using field data. Values of *RMSE* were between 0.08 and 0.1, and 0.05 and 0.06 for pressure and residual chlorine, respectively. The examination of water distribution system (WDS) during peak demand hours indicated that the pump unit's capacity could not meet the high-water demand, resulting pressure loss with values between 0.1 and 2 bar. Simulated residual chlorine levels ranged between 0.45 and 0.8 ppm.