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Research Article:

Hydraulic Assessment and Improvement of Water Distribution Network in Majidbag Quarter, Sulaymaniyah City, KRG, Iraq

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Abstract

This paper evaluates the hydraulic performance of a water distribution system in Majidbag Quarter in Sulaymaniyah City under peak summer conditions. The network was analyzed as having some supply and pressure shortages which had been increased by population and limited sources with limited supply, which was calculated by a model created in ArcGIS and simulated in WaterGEMS. The model calibration was conducted using the Darwin Calibrator tool and the R^2 value of 0.994 was obtained between simulated results and field taken pressures. In year 2025, demand conditions in the analyzed version of the existing system showed serious problems: in terms of pressure, 35 of 97 nodes were below the minimum required pressure standard (15 m), and many of the pipes analyzed as the distribution ones had flow velocities in which 37 pipes exceeded 2 m/s and 7 pipes operated below the recommended lower limit of 0.2 m/s. In order to improve these gaps, three scenarios of improvement were simulated. The initial two cases were to increase main pipeline diameter to (300 mm and 350 mm) that resulted in an increase in overall pressure but could not fix velocity issues in the distribution network. The third and most useful condition was a combination of a detailed network zoning strategy, adding an extra transmission main, and applying parallel piping in bottlenecks. This combined strategy was effective in bringing all nodal pressure up to the standard range of (15-60 m) and considerable enhancement of velocity profiles, 118 of 126 distribution pipes were working within the optimal range of 0.2-2 m/sec. The paper concludes by noting that although pipe enlargement has some positive advantages, reorganization of the system plan using zoning and parallel conveyance is the key to attaining sustainable hydraulic performance and stable and reliable supply of water.

1. Introduction

A water distribution system is a compulsory infrastructure system that has made communities access to water. It comprises of reservoirs, tanks, pumps, pipes, valves and other fittings which convey water either in their source or in their treatment stations to the consumers. The major role that the network plays is to fulfill demand without compromising the pressure and the desirable water quality [7]. Over 95 percent of the Sulaymaniyah city population have access to the public water network that is both

intermittent (IWS) and continuous (CWS). Most of the distribution is covered by the IWS, however, low water pressure is a prevalent problem, which compels residents to store water to use in day-to-day activities. The supply frequency is different in different neighborhoods whereby some are supplied every two or three days [23]. In 2025 the city experienced a major water crisis with the falling Sarchnar level, drying wells, and dwindling flows into the Dukan Dam, which decreased down to 24 percent of its capacity, the

lowest in 20 years. This made the delivery of water cut down to every five days due to reduction in the water production capacity from the sources [Sulaymaniyah Water Directorate]. Mathematical models of water distribution networks are created to solve these challenges. The models have been useful in designing and running systems and one can simulate various situations using them. The current computer-based hydraulic simulators are good and offer efficient resolutions as compared to the earlier days where the engineers were forced to read the results manually since computer graphics were not as well developed as they are now. The models can be combined with software nowadays (GIS, CADD, SCADA, CIS, CMMS and AMS can be named) and improve the process of data sharing and management. [13]. Computer models are capable to represent real-world networks and are useful in the study of hydraulic behavior. By using the underlying principles or principles of conservation of mass and conservation of energy they compute pressures and velocities very fast and correctly thus are essential in the management of water supply system [20]. The use of computer software in the discovery of various solutions of water distribution networks is well documented. Scholars have been exploring the performance of water distribution networks effectiveness in different ways with various computer software like OPTIGA, EPANET, WaterCAD and WaterGEMS [26]. An analysis of the literature shows that there are uniform results in relation to performance and analysis of water distribution networks (WDNs). In general, the researchers have found that there are severe hydrostatic shortages, and most systems fail to support proper pressure levels. Among them are low pressure common at all points in the network [11, 24], negative pressure at nodes [16, 17, 30] and a drop in pressure at remote locations or elevated points in the network [2, 9]. Also, the velocities of flow in pipes are often not within acceptable standards, either too slow in lateral lines [6, 11, 23], which causes sedimentation problems or too high in certain mains, which increases leakage and bursting risks. Such shortcomings are normally explained by higher demand, undersized pipes, inadequate pump capacity, system losses and bad network geometry like dead ends [6, 16, 26]. In order to identify such problems, researchers are inclined on hydraulic modeling that is always a useful and precise tool when it is appropriately calibrated. Calibration with field data is also commonly performed at a high level of accuracy, and several studies have indicated good correlation (R^2 values of 0.93 to 0.99) [11, 15, 19]. There are both steady-state and extended period simulation (EPS) models applied in order to test systems with different demand conditions as well as peak demand and future projected demand conditions [5, 9, 30]. The utility of such future projections is

especially those that indicate that networks that are sufficient to meet present demand [21, 27] will not cope with higher future loads, and hence the usefulness of models in long-term planning. The literature adheres to a standard set of strategies of optimization and rehabilitation in response to identified problems. The typical solutions comprise the expansion of the system [30], the increase of the length of pipes [11, 16], the addition of parallel pipelines, and the modernization or the rescheduling of pumps in order to increase the pressure and capacity of the system [3, 6]. Extending the head of a reservoir or introducing new water sources is also often recommended to serve the demand [5, 9]. Moreover, the modeling software with advanced optimization modules is effectively employed to automatize the process of the design parameters improvement [23]. Finally, the overall results confirm that computer-based hydraulic modeling is a necessitating, time-saving, and cost-efficient tool to assess, design, and optimize water distribution systems to provide reliable and sustainable service [19, 21, 24]. The aim of this research is to determine the hydraulic performance of a water distribution system (WDS) at the peak period in the summer season, i.e., the parameters of pressure and velocity. In ArcGIS, field data in Excel and AutoCAD is used to create a network model, which is simulated in WaterGEMS to determine the areas of low pressure or high flow velocity relative to the normal ranges. In order to overcome these problems, two scenarios of improvements are discussed which include taking of main pipelines to be enlarged, and a more advanced scenario where the concept of sub zoning and the introduction of additional transmission main and parallel piping is introduced in selected regions. The paper ends with a scientifically based and practically viable intervention plan that could improve the work of the system.

2. Materials and Methodology

2.1 Description of study area

Sulaymaniyah City is located in the North-East of Iraq, geographically it is located between $45^{\circ} 26' 27''$ East and $35^{\circ} 33' 42''$ North as shown in figure 1. It has an elevation generally ranging from less than 830 m to more than 1050 m above mean sea level [22]. Majidbag Quarter, which is currently being worked on, is identified as one of the most problematic zones in Sulaymaniyah City. The total length of the network is 11,743.23 m. "Based on the official records provided by the Mukhtar of Majidbag, the network serves approximately 4825 people and 960 houses" and the use of Google Earth pro imagery facilitated clear visualization of block separations and building distributions as shown in figure 2. Additionally, it consists of a reservoir with a 4800 m³ capacity and the completed network pipe layout and reservoir locations developed in ArcGIS are shown in figure 3.

2.2 Population

Sulaymaniyah City's water distribution networks (WDNs) have recently been struggling with poor and insufficient water pressure. This issue stems from a rapid annual population growth rate of about 2.64%. Census data shows the city's population was 604,000 in 2012 and is forecast to hit 847,000 by the end of 2025 [31]. Such growth has resulted in haphazard urban development and numerous unauthorized taps into the public water main. Consequently, the demand for water now surpasses the supply. The study area was carried out by the use of a spatial analysis that was backed by field verification to estimate the population. Initially boundaries were delineated using Google earth imagery and also assisted in subdivision of the area into separate blocks in accordance to observable urban lines. The residential and non-residential structures, including commercial and institutional ones, were identified within every block. It was then followed by on-site visits to ensure that the number of buildings was confirmed, their types verified, and residential and non-residential buildings differentiated. Based on the data, which was recorded, and which Sulaymaniyah Security Directorate provided, the total amount of populations, which were living in the area that was targeted, was estimated to be 4825 people in April 2025

2.3 Daily Water demand estimation

In order to evaluate the water network of the study area, the demand was classified into domestic, non-domestic (institutional, commercial and firefighting), and water loss. The domestic demand was considered as household consumption and the non-domestic demand comprised the institutional and commercial plants. Moreover, the water loss allowances were included into the general network analysis.

Domestic water demands (DWD):

Since the KRG has not issued official documents on the water distribution system, the Sulaymaniyah Water Directorate relies on practical experience. They estimate household average water demand at 250 l/c/d. Supporting studies include Kossay (2011) [18], who reported summer water demand in Mosul city ranging from 171–299 l/c/d [4], and Rabie (2025), who found consumption in Baghdad city, between 180–285 l/c/d [25]. Based on these references, this study adopts 250 l/c/d as the standard for domestic water use, with demand values calculated using the following eq.(1).

$$\text{DWD} = P \times \text{per capita domestic demand} \dots\dots\dots (\text{eq.1})$$

$$\text{DWD} = 4825 \times 250 = 1206250 \text{ L/day}$$

Non-Domestic water demand (NDWD):

Non-domestic water consumption covered the needs of institutional facilities such as mosques and schools, commercial establishments including various shops. The calculated NDWD is (50700 L/day) based on the eq. (2) below for different items:

$$\text{NDWD for facility} = \text{No. of facility} \times \text{Water consumption} \times \text{No. of capita} \dots\dots\dots (\text{eq.2})$$

Firefighting water demand (FFD)

According to the Egyptian code, the fire duration, and the amount of water demand of firefighting relies on the population of the area. In this study, water demands must be calculated for duration of 2 hours since the inhabitants of the study area were 4825 that is less than 10000 [14].

$$\text{FFD} = 2\text{hr} \times 20 \text{ L/s} \times 3600 \text{ s/hr} \times 24 \text{ hr/day}$$

Water losses (WL)

For this study, water loss was quantified by applying a standard assumption of 10% to both domestic and non-domestic demands, consistent with methodologies reported in prior research [8]. The calculation was performed using the eq. (3) presented below to derive the estimated volume.

$$\text{WL} = \% 10 \times (\text{DWD} + \text{NDWD}) \dots\dots\dots (\text{eq.3})$$

$$\text{WL} = 0.1 \times (1206250 + 50700) = 125695 \text{ L/day}$$

In this study, the total average water demand was obtained by summing all previously defined categories of demand using eq. (4).

$$\text{TWD} = \text{DWD} + \text{NDWD} + \text{FFD} + \text{WL} \dots\dots (\text{eq.4})$$

$$\text{TWD} = 1206250 + 50700 + 6000 + 125695 = 1382645$$

L/day

2.4 System Description and Components

The network is supplied by a single source, the Hawarabarza reservoir, which operates via gravity. The tank has a base elevation of 994 meters above sea level and a storage capacity of 4,800 m³. The existing WDN is a series of pipes of different materials including high-density polyethylene (HDPE), and ductile iron (DI), and diameter ranging between 75 mm and 400 mm. The network modeled has 97 junctions and 137 pipes, properties summarized in Table 1. The developed hydraulic model in WaterGEMS, representing pipes, is illustrated in Figure 4.

2.5 Design Specifications and Operational

Safeguards

The pipeline system allows a standard operating pressure of up to 15 meters of water column, but it is pressure-tested to allow 60 meters. There are a number of features that are added to facilitate functional integrity and maintenance. To avoid the occurrence of vacuum conditions, air valves are provided at all high points to make sure that the minimum pressure does not drop below 3 meters of negative pressure when compared to the atmospheric pressure. Washouts are placed at the low points in order to allow the flushing and draining of sediments. The gate and butterfly valves have been positioned in such a manner so that it can be used to isolate the pipeline sections to facilitate repair and maintenance work.

2.6 The Geographic Information System

Geographic Information Systems (GIS) is a vital instrument to spatial and statistical analysis of water resources, which promotes the efficacy of management to a significant degree [28]. GIS enables faster and better decision-making in the planning of hydraulic infrastructure of urban areas by offering an integrated

platform of visualization of both source data and model outputs. Nevertheless, the creation of a GIS framework and procurement of data necessary to complete management of water services is an intricate, expensive, and time-consuming task [1]. It is therefore a well-known fact that the combination of GIS with the hydraulic simulation models of the water distribution systems would be required to attain optimum outcomes in management. This synergy is essential, since the connection of GIS applications to external analytical models significantly increases the effectiveness and management of water distribution systems as a whole [29]. The everyday use of this technology is reflected in the task like the network layout design which is achieved with the help of GIS [2].

2.7 Bentley WaterGEMS:

Water GEMS is a hydraulic software that is applicable in water distribution modeling and optimization, has advanced interoperability, geospatial model building, optimization, and asset management tools. It eases the planning and optimization of the distribution networks to supply water continuously and intermittently. WaterGEMS is applicable in managing water system data, time-series hydraulic outcomes, present and prospective situations, and fundamental infrastructure data in a GIS setting [23].

2.8 Pressure gauge

Mechanical pressure gauge is a device to measure pressures of some points of a target water network. In this research, this was used to take pressure measurements from home faucets for water network calibration.

2.9 The elements of the existing WDS

The current water distribution system (WDS) comprises of 137 pipes, which are divided into transmission, main and distribution lines. In order to determine performance, pipe velocities were measured individually in each category in accordance with the set velocity limits. The transmission pipe is made up of four pipes all of which are tested in terms of their velocity performance. This was found to be appropriate because flow velocities were not higher than 3m/sec recommended in literature to minimize the chances of pipe bursts and leakage. The primary pipeline system is composed of eight pipes, and all of them were tested in terms of velocity. The literature gives previous studies that suggest that the pipe velocities should not be more than 3m/sec [10]. The distribution network has 125 pipes all of which were tested in terms of velocity performance. The suggested range of operation velocities is 0.2-2 m/sec.

2.10 Model Calibration

The process of a water system in which observed data of field and results of models are in comparison are entitled calibration. In case of necessity, the acceptable agreement of both data is obtained by adjusting the input data of the network. This process involves

altering demands of nodes, changing the roughness of pipelines, varying the characteristics of pump operation and modifying other data of model which have effects on obtained results. For this study, calibration merely focused on water demand changes not C factor, therefore C value of 140 for ductile iron and 150 for HDPE remained constant. Calibration junctions were randomly laid out in the WDN model, two of them having a high elevation and the remaining being downstream and located at a distance to the tanks. Four additional junctions shown in figure 5, JC1, JC2, JC3, and JC4, were introduced at pipes 19, 42, 72, and 115. These junctions were located close to the sample points, and they were directly hooked to the house faucets so that the calibration process reflected on the actual usage of water by consumers. In order to enhance the model, WaterGEMS Darwin Calibrator was used to determine the best solutions by scaling the total water demands and adjusting the chosen 81 junction water demands of the WDN model. The tool was useful in reducing the difference between observed and estimated hydraulic grade lines (HGL) at JC1, JC2, JC3 and JC4. As a result, the model was significantly calibrated since the obtained error at each JC ranged between -0.36 and 0.35 that fall in ± 1.4 m as depicted in figure (6). The calibration process proved highly effective, as the coefficient of determination (R^2) reached 0.994, as shown in Figure (7), indicating a strong agreement between the model's simulated pressures and the field-measured pressure data.

3. Results and Discussion

The water network in the study area was designed before 2015 and put into operation that year. Initially, it provided enough water throughout the seasons. Over time, however, population growth and the aging of pipes and components reduced its capacity, especially in summer when demand is highest. Consequently, extensive measures are being implemented to rehabilitate the system so it can meet essential water demand. In the summer of 2025, the water distribution system was intermittent, and it offered an average supply of approximately 1.5 hours per day (equivalent to 3 hours in every two days). Consequently, an estimated peak daily demand of 1,382,645 liters had to be met within this restricted supply period. Existing WDS Hydraulic simulations were conducted in steady-state conditions using demand data from summer 2025. Despite the system's intermittent water supply, the computed demand rates were applied directly to the hydraulic model. The assessment focused on nodal pressure, and pipe flow velocity, which were selected as the primary indicators of system performance during peak demand in summer 2025. For analysis the nodal pressure, the pressure of the water network was assessed at 97 junctions during the summer 2025 daily demand, and nodal values compared against the

standard limits of 15–60 m to check overall compliance. The classification results are summarized in Table (2). The analysis in Table 2 indicates that none of the junctions exceeded the maximum pressure limit. Nevertheless, 35 junctions recorded pressures which were below the minimum requirement, while 62 fell within the acceptable range, which illustrates constricted pressure distribution throughout the network at the time when the network is at peak demand during summer. The spatial distribution of nodal pressures is shown in figure (8) by use of WaterGEMS; the colors green represents pressures at or below 15 m and blue denote values within the acceptable range up to 60 m. Four transmission pipes were tested in terms of velocity performance when the highest demand of summer 2025 is reached. It was found that all the pipes were working within the accepted velocity limits as none of them had gone above the maximum limit of 3 m/sec. The results, as shown in Table (3), show that all the pipes-maintained velocities as low as 3 m/sec. Out of the eight main pipes assessed, only one met the acceptable velocity limit, while the remaining pipes exceeded the recommended maximum of 3 m/sec as presented in Table (4). Previous studies have suggested that maximum main pipe velocities should not exceeded 3 m/sec [10]. Distribution pipes showed varied velocity performance. Of the total, 81 operated within the accepted range, 7 fell below 0.2 m/s, and 37 exceeded 2.0 m/sec. The analysis results are summarized in Table (5) Velocities lower than 0.2 m/sec may lead to sedimentation and deterioration of water quality, while velocities above 2.0 m/s can increase head losses and impose hydraulic stress on the network [19]. Figure (9) illustrates the spatial distribution of water velocities across the network. Using WaterGEMS, color coding and labeling were applied to represent all velocity values. Pipes with velocities at or below 0.2 m/sec are shown in green, those exceeding 2 m/sec are marked in red, and acceptable velocities between 0.2–2 m/sec are indicated in blue. Generally, pressure in the pipes of the mainline remained within acceptable limit throughout the operation of the network, even when flow velocities dropped below or exceeded the permissible 3 m/sec limits. But it can lead to inefficiencies when long excursions of this range, such as non-uniform pressure distribution, energy consumption increase, and structural strain of the pipes may occur. In order to maintain constant pressure and improve hydraulic performance, it is recommended to control flow rates by adjusting pipeline diameters. These adjustments aid in maintaining the velocities at the permissible limit. To achieve this, the following scenarios were conducted. In the first scenario, the diameter of the existing main pipes changed from 250 mm to 300 mm in diameter. In general, the entire improvement in terms of pressure at whole junctions

within the network could be obtained. Besides, reduction in water velocity was observed in the available main pipeline, which is an indicator of capacity enhancement in the transport of fluids. On the other hand, there were no variations in the velocities in the transmission and distribution pipes, compared to the already existing network pipes. Although the water velocity of the main pipes was reduced it was still out of the acceptable range. Therefore, it requires further enhancement. Based on the results of the first scenario, the second scenario was conducted by enlarging the existing main pipes diameter further from 250 mm to 350 mm based on commercially available pipe sizes used and approved by directorate of water in Sulaymaniyah. In general, enlarging the existing main pipe's diameter to 350 mm resulted in a significant improvement of pressure levels throughout the network. The velocity within the main pipe was further diminished, achieving acceptable parameters. Nevertheless, the velocities in transmission pipeline and distribution pipes remained unaltered, similar to first scenario. This indicates that merely enlarging the existing main pipeline, even to a greater diameter, does not adequately affect velocity conditions within the distribution network. Increasing the diameter of the main line reduced friction loss and improved pressure, but water velocity in the distribution pipes remained unchanged because the system relies on a single main line. Thus, enlarging the main line alone is insufficient to redistribute flows effectively. To enhance performance, a zoning strategy was adopted to divide the network into hydraulically independent zones, aiming to reduce dependence on a single supply line and alleviate upstream hydraulic stress. Each smaller zone is supplied by one single pipeline. Consequently, a new scenario had to be developed. Third scenario was formulated for improvement strategy, including zoning the layout of distribution pipes, adding one extra main pipeline, and implementing parallel piping method, the analysis results are shown below:

The pressure performance of the scenario model was evaluated at 97 junctions under the daily water demand in summer 2025. The PRVs on the existing main line and added main line were set to regulate the system pressure. The evaluation of whole nodal pressures was carried out based on the adopted pressure limits of 15–60 m. Table (6) presents the pressure classification in terms of both percentages and number of junctions. The findings demonstrated that the whole network pressure satisfied the adopted pressure criteria. The spatial distribution of nodal pressures across the network is illustrated in Figure (10). The color coding and labeling processes were carried out for the entire junction pressures in WaterGEMS, the blue color shows acceptable pressure values between 15–60 m. Distribution pipes, in third scenario, show a wider range of velocity behavior. The total number of the

distribution pipes is 126 which were assessed for velocity analysis. 118 pipes fell within adopted range, whereas 7 pipes operated below the minimum velocity and 1 pipe exceeded the maximum recommended velocity. The velocity analysis is summarized in table (7). Furthermore, the spatial distribution of water velocity across the network is illustrated in Figure (11). The color coding and labels processes were carried out for the entire pipe velocities in WaterGEMS, the green color shows velocity values equal or below 0.2 m/sec and the red color indicates the values of velocities above 2 m/sec while the blue color shows acceptable velocity values between 0.2-2 m/sec. Overall, the significant improvements observed in the third scenario confirmed the effectiveness of the integrated network restructuring strategy. Zoning enabled better flow redistribution and reduced dependence on a single supply route, while the supplementary main line provided sufficient conveyance capacity to meet peak demand. In addition, the parallel pipes helped alleviate flow concentration and excessive velocities within the heavily loaded distribution system. However, the addition of new pipes caused pressures at several downstream junctions to exceed the upper limits. To address this, pressure reducing valves (PRVs) were installed on the main pipes within the water model, ensuring that the water distribution system operated within the adopted pressure range.

4. Conclusion

The study evaluated the hydraulic efficiency of the existing water distribution network in the Majidbag Quarter during peak demand season and proposed practical strategies for its rehabilitation. Founded on this analysis, the following conclusions can be drawn:

- 1) The result analysis of existing network showed that Pressures of some junctions of the water network did not operate under adopted pressure limit. Their values experienced low values, demonstrating inadequate severe level in those sections. High value of water velocity was noticed which located in the existing main line (main pipes) as well as the upstream part of distribution pipes that may lead to burst the pipes and leakages.
- 2) The occurrence of deficiency in pressure and velocity in the system was primary related to factors such as undersized pipe diameters and inadequate flow distribution due to existing large network layout. Therefore improvement strategy is necessary.
- 3) The sequential improvement scenarios showed that diameter enlargement of the main pipeline (Scenarios 1 and 2), improved overall system pressure, but did not resolve velocity inequalities across the distribution network.
- 4) Third scenario was the optimum alternative that offered the solution and it comprised of integrated structural redesign that incorporated network zoning, inclusion of a supplementary transmission main and

strategic parallel piping. This approach was effective to ensure that all the nodal pressures were within the acceptable zone (15 - 60 m) and the velocities were optimized, 118 out of 126 distribution pipes were within the recommended range of 0.2-2.0 m/sec.

5) The installation of pressure-reducing valves (PRVs) was necessary to manage the increased pressure in downstream sections caused by the upgrades.

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التقييم الهيدروليكي وتحسين شبكة توزيع المياه في حي

مجيدبك، مدينة السليمانية، إقليم كردستان العراق

المستخلص

تقيم هذه الدراسة الأداء الهيدروليكي لشبكة توزيع المياه في حي مجيدبك بمدينة السليمانية خلال ذروة الطلب الصيفي. وبالنظر إلى مشكلة الإمداد المتقطع ونقص الضغوط التي تفاقمت بفعل النمو السكاني ومحدودية المصادر، جرى تقييم الشبكة باستخدام نموذج مطور في برنامج ArcGIS ومحاكى في WaterGEMS. أُجريت معايرة النموذج باستخدام أداة معايرة داروين the Darwin Calibrator، وتم الحصول على قيمة R^2 قدرها 0.994. بين نتائج المحاكاة والضغوط المقاسة ميدانياً. أظهر تحليل النظام القائم تحت ظروف الطلب المتوقعة لعام 2025 مشكلات جوهرية؛ من حيث الضغط، إذ إن 35 عقدة من أصل 97 عقدة انخفضت عن معيار الضغط الأدنى (15 م)، كما أظهرت العديد من الأنابيب سرعات خارج النطاق المقبول، حيث تجاوزت 37 أنبوبية توزيع سرعة 2 م/ث، في حين انخفضت 7 أنابيب عن 0.2 م/ث.

ولمعالجة هذه النواقص، جرى اختبار ثلاثة سيناريوهات تحسين. السيناريو الأول والثاني تضمنتا تكبير قطر الأنابيب الرئيسي (إلى 300 مم و 350 مم)، مما أدى إلى تحسين الضغط العام لكنه لم يعالج مشكلات السرعة في شبكة التوزيع. أما السيناريو الثالث والأكثر فاعلية فقد اعتمد استراتيجية شاملة تضمنت تقسيم الشبكة إلى مناطق (Zoning)، وإضافة خط ناقل إضافي، وتنفيذ أنابيب موازية في المقاطع الحرجة. وقد نجح هذا النهج المتكامل في رفع جميع الضغوط العقدية إلى ضمن النطاق المعياري (15-60 م)، كما حسن بشكل ملحوظ من خصائص السرعة، حيث عمل 118 أنبوب توزيع من أصل 126 ضمن النطاق الأمثل (0.2-2 م/ث).

وتخلص الدراسة إلى أن تكبير الأقطار يوفر بعض الفوائد، إلا أن إعادة هيكلة تصميم النظام بما يشمل التقسيم الشبكي والتوصيل الموازي يُعد ضرورياً لتحقيق أداء هيدروليكي مستدام وضمان إمداد مائي موثوق.

الكلمات المفتاحية:

واترجيمس، أرك جي أي إس، داروين كالبريتور، شبكة توزيع المياه، استراتيجية تقسيم المناطق.

Table (1): Specification of the existing pipes

NO	Diameter(mm)	Material	Numbers	Length (m)	Pipe function
1	75	HDPE	39	3,957.98	Distribution pipe (D.P.)
2	100	HDPE	44	3,081.80	Distribution pipe (D.P.)
3	150	HDPE	33	1,942.20	Distribution pipe (D.P.)
	150	D I	6	460.88	Distribution pipe (D.P.)
4	200	D I	3	147.11	Distribution pipe (D.P.)
	200	D I	2	19.06	Main pipes (M.P.)
5	250	D I	6	316.43	Main pipes (M.P.)
6	350	D I	2	750.51	Transmission pipes (T.P.)
7	400	D I	2	1,067.26	Transmission pipes (T.P.)
Total			137	11,743.23	

Table (2) Pressure percentage of existing water distribution

Pressure Limits	Number of Nodes	Percentage (%)
≤ 15	35	36.08
15 - 60	62	63.92
≥ 60	0	0
Total	97	100

Table (3) Velocity percentage of transmission pipes

Velocity limits (m/sec)	Number of pipes	Percentage (%)
≤ 3	4	100
> 3	0	0
Total	4	100

Table (4) Velocity percentage of main pipes

Velocity limits (m/s)	Number of pipes	Percentage (%)
≤ 3	1	12.5
> 3	7	87.5
Total	8	100

Table (5) Percentage of velocity result of distribution pipes

Velocity limit (m/sec)	Number of pipes	Percentage (%)
≤ 0.2	7	5.6
0.2-2	81	64.8
≥ 2	37	29.6
Total	125	100

Table (6) Pressure limits of junctions and its percentage

Pressure limits	Number of nodes	Percentage (%)
≤ 15	0	0
15-60	97	100
≥ 60	0	0
Total	0	100

Table (7) Velocity limits and its percentage of distribution pipes.

Velocity limits (m/s)	Number of pipes	Percentage (%)
≤ 0.2	7	5.56
0.2-2	118	93.65
≥ 2	1	0.79
Total	126	100

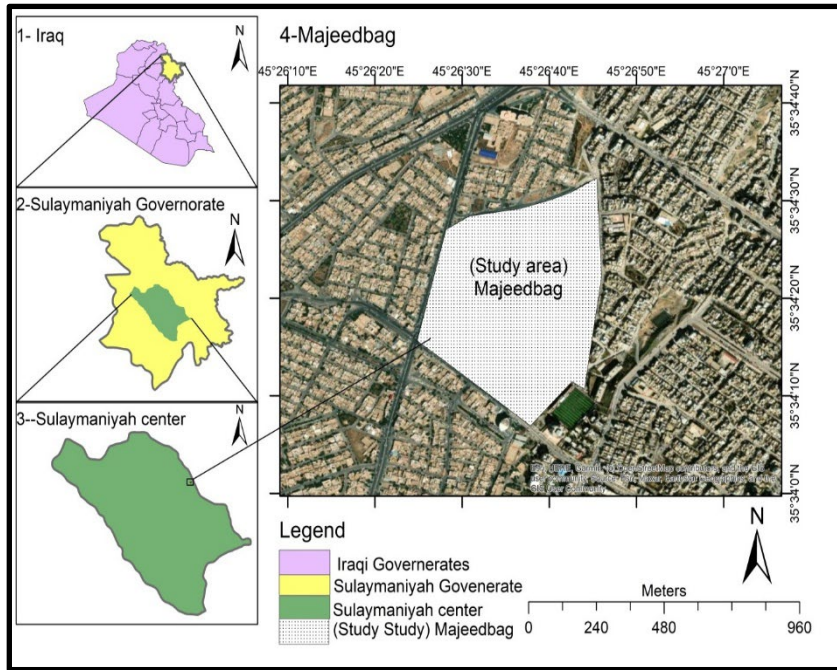


Figure: 1 The location of study area

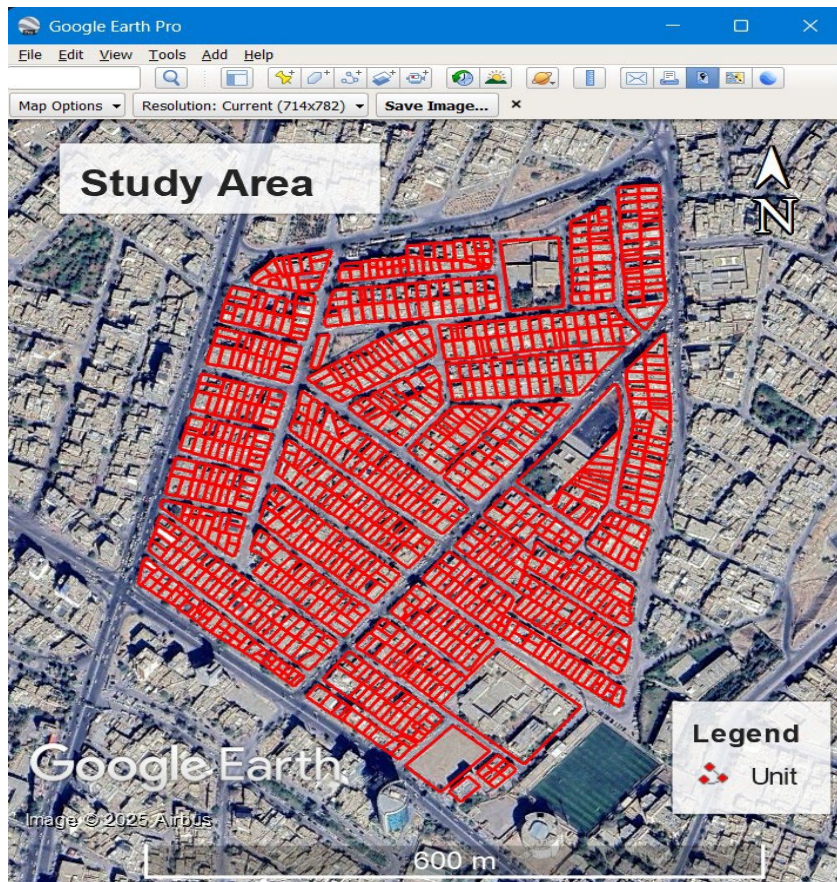


Figure: 2 Boundary of each block and units

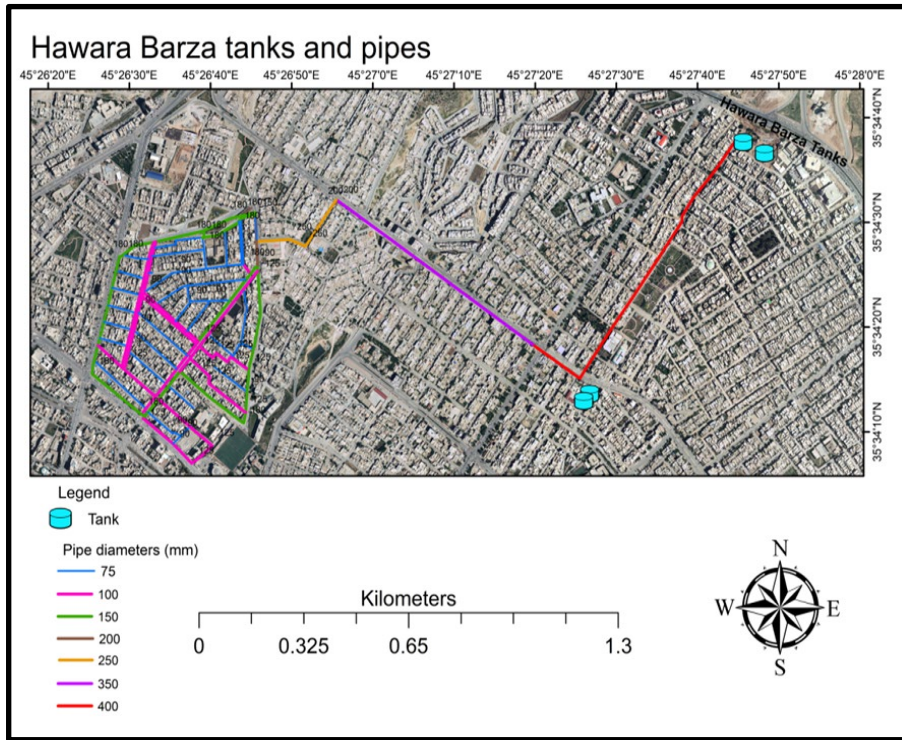


Figure: 3 Pipelines and the tanks (using ArcGIS 10.7)

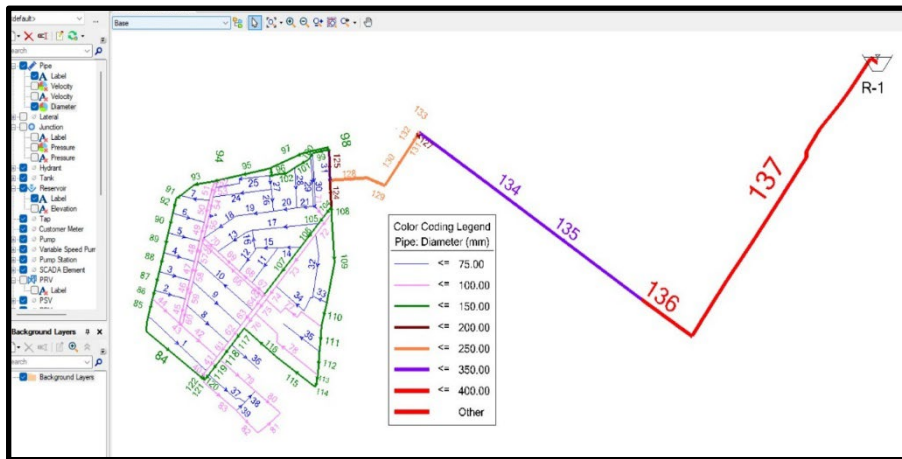


Figure: 4 Pipe labels and diameters of water model

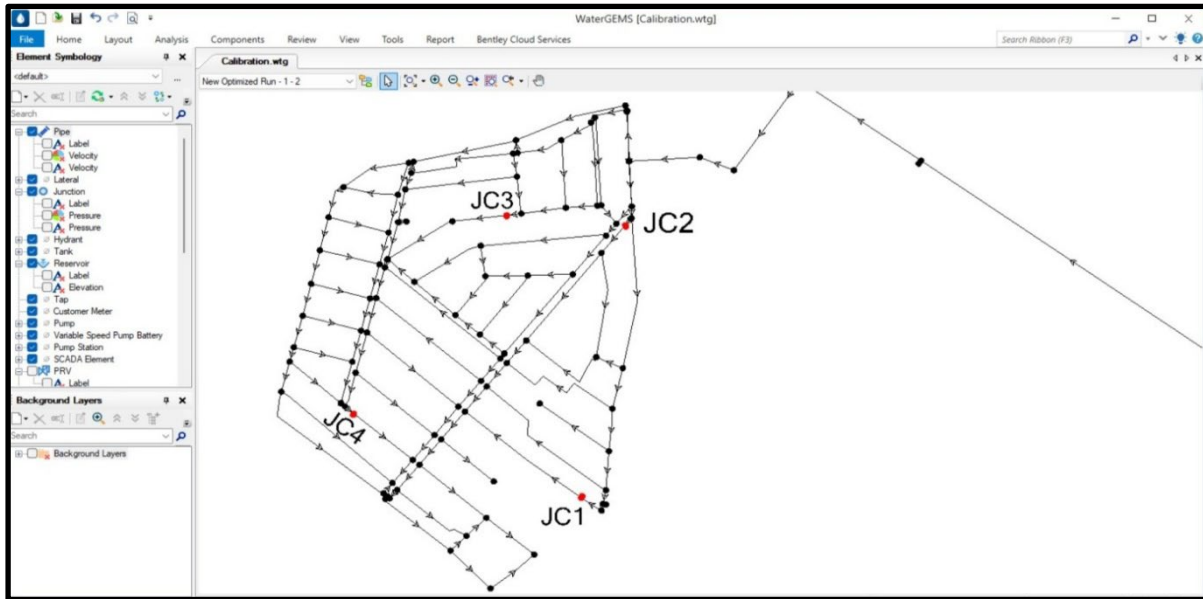


Figure: 5 Selected junctions for model calibration

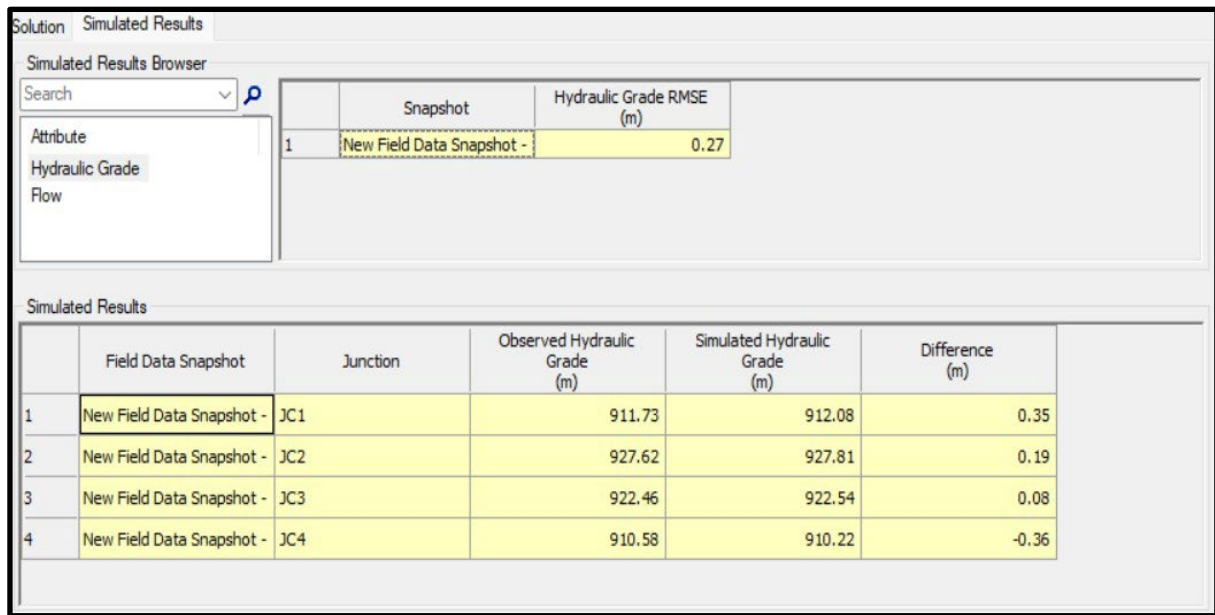


Figure:6 Model calibration using WaterGEMS

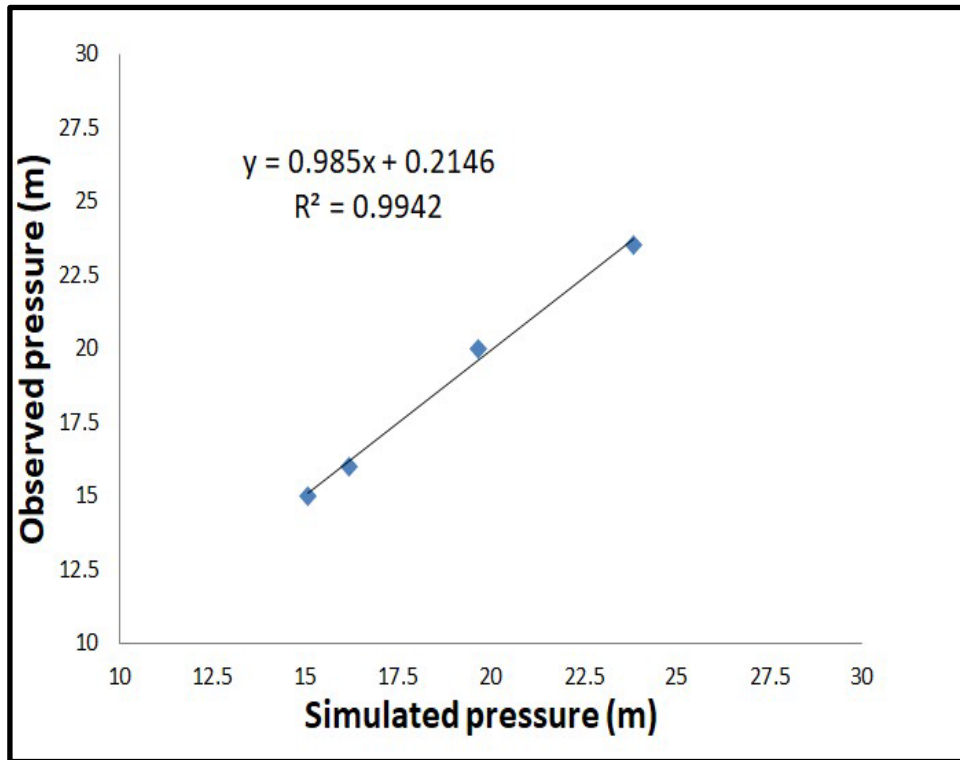


Figure: 7 Comparison between model's simulated and the field-measured data

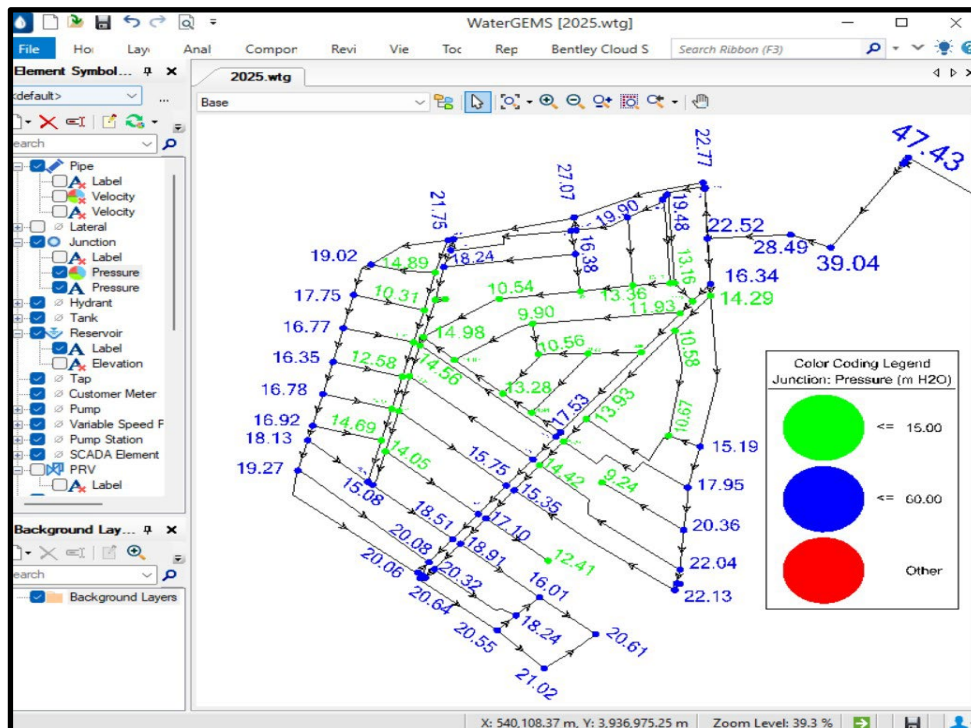


Figure: 8 presents the pressure results of the existing water network in summer 2025, generated using WaterGEMS.

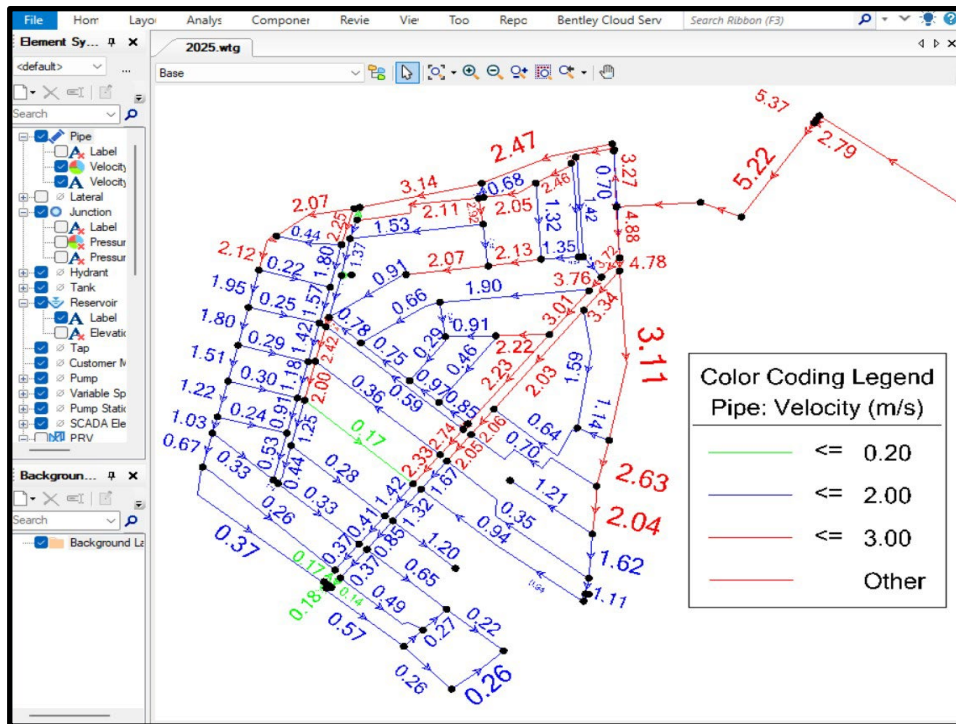


Figure: 9 Results of flow velocity assessment for distribution and main pipes in WaterGEMS

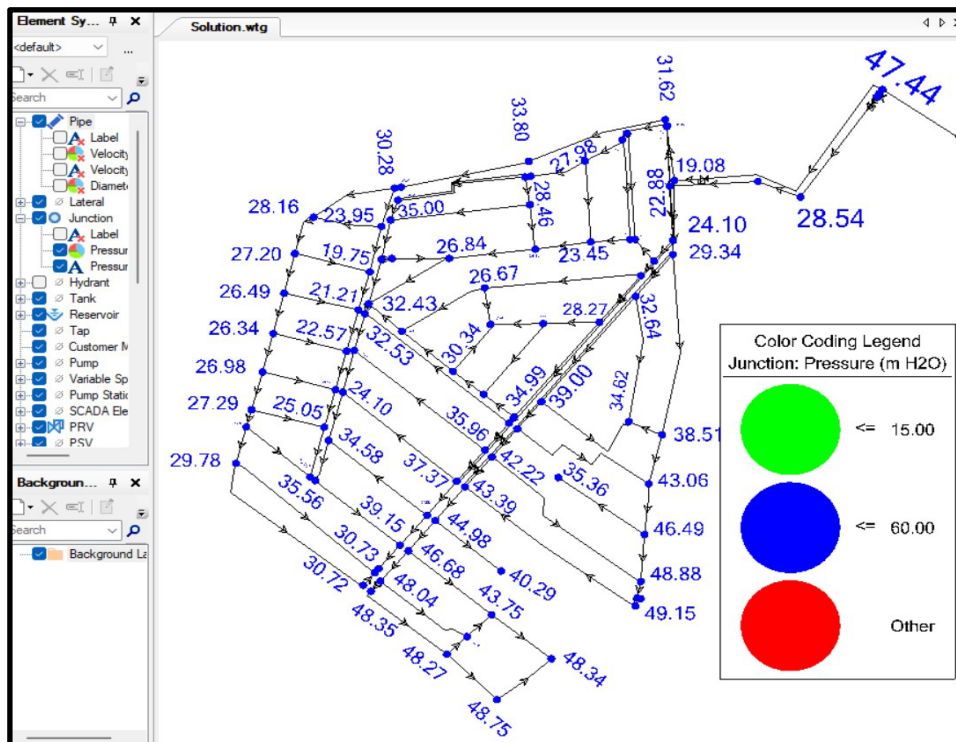


Figure: 10 Pressure results of scenario three (using WaterGEMS)

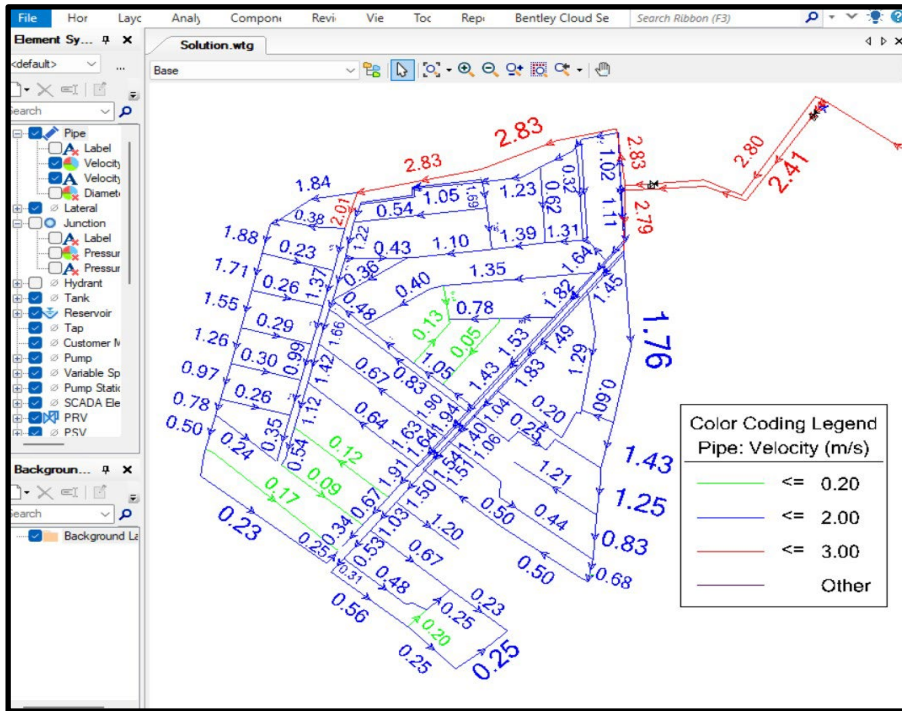


Figure: 11 velocity results of scenario three (using WaterGEMS)