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The estimation of the hydraulic model and the measured pressures of ground water reservoirs based on the water model were not taken into account

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Abstract

Nowadays, due to the complexity of water distribution systems and their large scale, their design, operation and maintenance require the use of optimal methods, which have become more important than in the past in improving their calibration. The most important issue in the simulation modeling of these systems is the consistency between the calculated and measured data. In the absence of the results of unaccounted water studies in the network, the use of statistical experimental methods is still needed as one of the main elements in model calibration. Based on this, the current research was conducted to investigate the effects of different patterns of unaccounted water based on the calculated water consumption patterns and to determine the optimal pattern of unaccounted water within the water distribution network covering the number of 6 ground reservoirs. The comparison of the statistical parameters showed that the use of the inverse model of the customers' consumption, which is not considered as a water model, for calibrating the hydraulic model of the distribution network, provides more acceptable limits for the closeness of the predicted values to the recorded values of the hourly output of the reservoirs, and therefore It is better to be used in studies related to planning and designs.

Keywords: water distribution system, unaccounted water, hydraulic model, pressure distribution, measured pressures
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1 Introduction

Today, with the growth of the urban population and the development of cities, water distribution systems have become very important. Considering the complexity of these systems and the large scale of decision-making in the analysis, design, operation and maintenance of the seta, the need for computer modeling of the seta has become more important than before. In general, water distribution networks are a very complex combination of thousands of pipes, nodes and connections, however, the number of measurements performed is reduced to only a percentage of the entire network, and this makes the model calibration even in some cases not It may come close. The most important issue in the simulation modeling of sets is the consistency between the calculated and measured data [5].

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Optimizing the network and detecting the fault and location in the cells are two measures that can be taken to reduce water and energy losses. Many of these techniques require a good calibration model to produce reliable results. Model calibration is the adjustment of network parameters to reduce the error in predicting the results [7].

Since the data used to create hydraulic models are mainly extracted from different sources such as geographic information system (GIS), database, customer water bill archive and data collection and control system (SCADA), therefore the total estimate Consumption becomes quite complicated. In such a situation, a multivariate solution may be considered using optimization procedures to calculate the consumption, because the exact solution cannot be obtained based on the measured flows and changes in the amount of water in the reservoirs. Therefore, the problem arises that the modeled flow will not be exactly equal to the measured water flow. Basically, the amount of unaccounted water in a distribution system can be determined by conducting water balance studies in the system or in a measurement enclosed area (DMA). In addition, estimation of unaccounted water using statistical techniques has been reported by various researchers [11].

Existing studies show that in the absence of DMA survey results that indicate reliable amounts of unaccounted water in the network, the use of recommended and empirical statistical methods can be the only available option. But it should be noted that in big cities, the network and type of consumption (domestic, commercial, industrial) is very complex and the pattern of consumption in each area depends on the mentioned conditions and is different from other areas. Therefore, the generalization of the recommended mathematical formulas and methods still did not increase the accuracy of calibration in water distribution network simulation models and therefore did not provide the possibility of reliable prediction and estimation for development, modification and reconstruction plans [1]. The current research, in order to investigate and improve the recalibration of the continuous model of the water distribution network, by introducing, investigating and implementing an optimal integrated experimental approach of the unaccounted water pattern, was carried out and the effects of different unaccounted water patterns based on the usage pattern The calculated water based on the Seta model has been evaluated by comparing the output of the model with the actual conditions in the studied network.

2 Theoretical foundations and research background

2.1 Water distribution networks

Some infrastructures and buildings are very essential for communities. Water supply systems are one of these structures, which are generally defined as multi-purpose networks for distribution, transmission, storage and distribution of water for drinking, commercial, industrial and agricultural uses, as well as for some public needs such as Firefighting and green spaces are among the general functions of distribution networks. The structures of water distribution systems are designed and built on the basis of providing water with sufficient pressure and making it available to subscribers. Today, water distribution networks have intertwined systems that require high investment to build, operate and maintain.

2.2 The main components of water distribution systems

Despite the size of pipes and network components and the complexity of water supply in the systems, they all have one basic purpose, which is to deliver water at the required pressure. These systems usually form an interconnected network by different components, each component performs a specific task/tasks. The main components of a water distribution system are shown in Figure 1.

2.3 Water distribution network calibration problems

The most important issue in the modeling of sediments is the consistency between the predicted model and measured data. To achieve this goal, it is necessary to calibrate the model through measurement data. The problems of recalibration of water distribution network have been presented in many researches. In general, the roughness of the pipes and the distribution of the total consumption in each node have been reported as effective factors in the calibration [7]. In 1988, Ormesby presented the calibration algorithm by defining the pressure reduction factor as a basic solution of explicit network equations. Also, the direct effects of pressure and current distribution in the network, which plays an important role in matching the real conditions, have been described by Datta in 1994. This researcher suggested the use of weighted least squares (WLS) using sensitivity analysis to solve the problem of the inverse effects of the roughness coefficient [3].

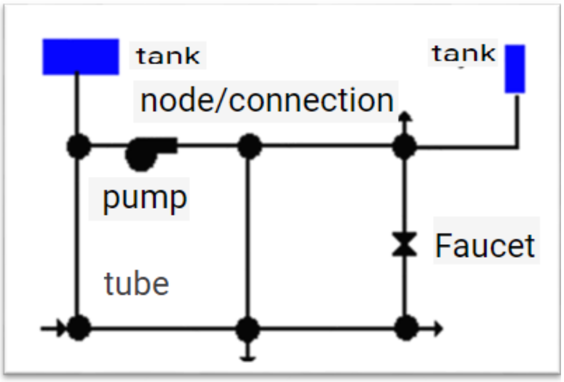

Figure 1: Main components of water distribution network

In another study, two consumption parameters and roughness coefficient were estimated using the WLS method based on Gauss-Newton minimization technique. In order to calibrate 1000 tubes in the Walters network, 90 pressure measurements were made for use in genetic algorithms. The results were evaluated well, although there was a 2 m difference in pressure estimation [16].

Consumption regulation is a common problem with an adverse effect on optimization. In 2010, Axela used calculated and measured weekly consumption to classify different households [1]. This classification allows the estimation of curves using combined Gibbs and Gaussian sampling. Both methods provided equations for network problems in which the number of measurements and the number of parameters required for estimation were similar. At the same time, there was a need to combine both stages of consumption estimation to have a convergent method.

2.4 Water distribution network calibration methods

Calibration methods based on optimization can be classified based on conditions, such as uncertainty parameters in existing conditions, which maximize design content information, and uncertainty in forecasts, which reduces the average forecast dispersion. In 2007, based on the formulation and solution of the equation for optimization, Kumar presented a genetic algorithm. Nevertheless, the least squares method is still the dominant method in the optimization of the seta model [13].

Evaluating calibration accuracy based on the ratio of observations to pressure loss predictions is older than evaluation based on the difference (distance) of observations from predicted pressures. This is because the ratio of observations to predictions shows which parameters need to be adjusted, and the index of the difference between observations and predictions for relatively smooth systems does not provide a special meaning.

2.5 Research background

Hydraulic simulation models are widely used by water planners, engineers, consultants and managers involved in the analysis, design, operation and maintenance of water distribution systems. To create efficient models, they must be recalibrated. Therefore, by determining the various parameters that are obtained in time periods, the inputs of a hydraulic simulation model, a logical relationship between the pressure and the measured and predicted flows in the network is presented. will do. Walski [15] was one of the first researchers who proposed to simulate a water distribution system by collecting pressure and flow data to calibrate the model. In 1994, Yu and Powell [17] presented the problem of installing a meter (flow meter) in a water distribution system with the aim of maximizing accuracy and minimizing cost as an equation in a dynamic analytical model using the covariance matrix of variables and presented decision tree techniques. After him, in 1994, Feriri et al proposed a method for selecting measurement points by evaluating relative sensitivities, taking into account calibration based on the roughness coefficients of nodes. In their proposed method, instead of the optimization equation for system calibration, they ranked the nodes according to their relative sensitivity in general. Later, a two-level optimization method to evaluate several calibration parameters was introduced by a researcher named Shader in 2000. In the external optimization loop, a simulation based on the annealing solution method was used to solve the maximization problem related to unknown values. In the inner loop, the calibration problem of standard minimization was solved by moving slope in pseudo-Newtonian method [4].

Kapelan et al. [8] linked the metropolitan algorithm (SCEM-UA) to the complete Epanet and used this model to solve a calibration problem with the least squares calibration criterion. In 2007, Colombo and Geistolis presented a meta-model approach to optimize water distribution system calibration, in which evolutionary polynomial interpolation (EPR) was used to solve the calibration problem [2]. The set of fuzzy series as a display of uncertain costs is presented in a modified hereditary finite element (GA) process to find optimal solutions in 2009 in another research. Also, Savich et al. [14] reviewed the calibration of the model and proposed a solution. suggested that the future approaches and challenges are taken into consideration regarding the hydraulics and the level of accuracy of the models [14].

3 Research methodology

3.1 Statistical community and network components

In this study, due to the availability of basic information including maps of the water distribution network and related reservoirs, information related to the location, type of consumption and periodic consumption of subscribers, a part of the water distribution network in the city of Tehran, including 7 reservoirs and the network under Their coverage was chosen as the statistical population for the research.

Water distribution networks (within the city) have components that from upstream to downstream include: storage tanks, main and semi-main distribution pipes, sub-distribution pipes and nodes (subscriber and consumer meters). The connection between the components of the distribution network is also established and adjusted by means of control valves, pressure and flow control valves and measuring meters. For all the mentioned components, their location is taken as a basic specification from the location of the maps from the archive of the water and sewage company of the studied area.

3.2 Water supply, distribution and consumption data

In this study, the data of water supply, distribution and consumption respectively in tanks, main pipes (outlet pipes of tanks) and customers' meters, in the form of a periodic database (at least for the last 5 years) from the water and sewage company of the studied area. Received and monitored and processed.

3.3 Data collection and collection tools

In this study, the desired data is collected in dwg and shp formats as a computer file on a CD or computer peripheral memory. For water supply, distribution and consumption data, the desired data is collected in the form of Excel spreadsheets or the QSL Server database by referring to the Water and Sewerage Company's Operations Subscriber Affairs Unit. In addition, for the lack of information sections, especially in the consumption information section of subscribers, if necessary, a standard questionnaire form will be used based on the design criteria for urban and rural water transmission and distribution systems, management and planning organization.

3.4 The results of water reports are not considered

If unaccounted water studies have been conducted in the investigated area and their results are available, they will be used as control parameters in determining the accuracy of the proposed method to determine the effect of the unaccounted water pattern on the model calibration. Although the existence of the results of unaccounted water studies in the scope of the study is not mandatory for the present research, but at the same time, the availability of the results can play an effective role in improving the accuracy of the results of this research. Unfortunately, in this study, despite numerous follow-ups, access to these reports was not possible.

3.5 Research method

After selecting the study area and also verifying them, the desired model is based and after determining the characteristics of the network components and reservoirs, the characteristics of the subscribers are entered as consumption nodes in the specialized software used as the main elements and the basis of the hydraulic model. and then by determining the boundaries of the ranges according to the actual feeding conditions of each user of the reservoir, demographics and per capita consumption are determined and then the maximum per capita consumption for each reservoir on the day of maximum consumption and how it changes is extracted. Next, in order to extract the consumption pattern, after preparing the basic model, the amount of consumption and the consumption pattern on the day of maximum consumption for the nodes corresponding to each reservoir are introduced to the software and next to the diagram of the output from the reservoir to the network for the same day, the hydraulic conditions of the network model are defined. will be. Finally, the hydraulic model is prepared according to the introduction of the mentioned parameters, it is set up and the amount of consumption and feeding of the tank and the conditions of pressure and velocity in the pipes are prepared as the raw output of the model. Hazen-Williams pipes, which represent non-revenue water patterns (as a direct, inverse, average ratio of per capita consumption).

In this study, Excel or SPSS software will be used to verify information in statistical sections, including consumption, flow and pressure measurements based on the usual methods of statistical data control. Also, statistical factors such as $R2$ will be used in the evaluation and analysis of model outputs. As a complementary evaluation of the output of Water GEMs model, the existing situation and the modeling of pressure distribution maps overlap will also be used.

3.6 WetSpa model

WetSpa is able to perform spatially distributed calculations through the availability of spatially distributed data sets (digital elevation model, land use, soil and radar-based precipitation data) and GIS technology. Precipitation, interception, depression storage, surface runoff, infiltration, evapotranspiration, percolation, interflow, ground water flow, and water balance in each layer are the hydrological processes that are considered in the model. The total water balance which is considered for each raster cell is composed of a separate water balance for the vegetated soil, bare-soil, open water, and impervious part of each cell. This allows to consider the non-uniformity of the land use in per cell which depends on the resolution of the grid. A mixture of physical and empirical relationships is used to depict the hydrological processes in the model. The model can predict the peak discharges and hydrographs in each place of the channel network and the spatial distribution of the hydrological characteristics of each cell. Hydrological processes are represented in a cascading way. After the precipitation, incident rainfall first encounters the plant canopy which intercepts all or part of the rainfall until reaching the interception storage capacity and then excess water reaches the soil surface and may infiltrate in the soil zone, enter depression storage, or may be diverted as the surface runoff. Some of the infiltrated water percolates to the groundwater storage and the remained is diverted as interflow. Total runoff from a grid cell is computed as the summation of surface runoff, interflow and groundwater discharge. The root zone water balance for each grid cell is modeled continuously through equating inputs and outputs as follow:

$$D\theta/dt = P - I - S - E - R - F \tag{3.1}$$

where $D[L]$ implicate the root depth, $h[L3L-3]$ show the soil moisture, I, $[LT-1]$ is the initial loss consist of interception and depression storage, $S[LT-1]$ is the surface runoff, $E[LT-1]$ is the evapotranspiration from the soil, $R[LT-1]$ is the percolation out of the root zone, $F[LT-1]$ implicate the interflow, and t is the time $[T]$. The surface runoff is computed by a moisture-related modified rational method with a runoff coefficient dependent on the land cover, soil type and slope:

$$S = C(P - I)(\theta/\theta s)a \tag{3.2}$$

where: θs = saturated soil moisture content $[L^3L^{-1}]$, Cr = potential runoff coefficient $[-]$ depending on slope, land use and soil type, and α = empirical parameter $[-]$. Exponent $\alpha[-]$ in the formula is a variable reflecting the effect of rainfall intensity on runoff generation.

$$\partial Q/\partial t + ci\partial Q/\partial x - di\partial^2 Q/\partial x^2 = 0 \tag{3.3}$$

where $Q[L^3T^{-1}]$ implicates the discharge, $t[T]$ shows the time, $x[L]$ shows the distance along the flow direction, $c[LT-1]$ is the location dependent on the kinematic wave celerity, is interpreted as the velocity by which a disturbance travels along the flow path, and d $[L^2T^{-1}]$ is the location dependent on the dispersion coefficient, which measures the tendency of the disturbance to disperse longitudinally as it travels to the downstream. Assuming that the water level gradient equals the bottom slope and the hydraulic radius approaches the average flow depth for overland flow, c and d can be approximated by $c = (5/3)v$, and $= (vH)/(2S0)[12]$, where $v[LT^{-1}]$ is the flow velocity computed by the Manning equation, and $H[L]$ shows the hydraulic radius or the average flow depth. An approximate solution to the diffusive wave equation in the form of a first passage time distribution is applied [10]. That relates the discharge at the end of a flow path to the available runoff:

$$u_i(t) = \frac{l_i}{2\sqrt{\pi d_i t^3}} \exp\Big[-\frac{(c_i t - l_i)^2}{4d_i t}\Big] \tag{3.4}$$

where $U(t)[T-1]$ implicates the flow path unit response function, serving as an instantaneous unit hydrograph (IUH) of the flow path, which makes it possible to direct the excess water from any grid cell to the outlet of the basin or to the any downstream convergent point, $t0[T]$ shows the flow time, and $\sigma[T]$ is the standard deviation of the average flowtime. Two parameters $t0$ and σ are spatially distributed and can be obtained through integration along the topographic determined flow paths as a function of flow celerity and dispersion coefficient.

$$t_0 = \int c^{-1} dx \tag{3.5}$$

$$\sigma = \int (2d/c^3) dx. \tag{3.6}$$

As the groundwater movement is much slower than the surface water and near surface water system movements and the understandings about the bedrock is little, groundwater flow is simplified as a lumped linear reservoir in small GIS derived subwatershed scale. With considering the river damping effect for all flow components, overland flow and interflow are directed firstly from each grid cell to the main channel, and are joined with groundwater flow at the outlet of the subwatershed. Then the total hydrograph is routed to the outlet of the basin by the channel response function derived from Equation (3.4). The amount of total discharge is sum of the overland flow, interflow, and groundwater flow, and is obtained by convolution of the flow responses of all grid cells. One advantage of this approach is allowing to the spatially distributed runoff and hydrological parameters of the basin for using as inputs for the model. Inputs of the model consist of digital elevation data, soil type, land use data, and measured climatological data. Stream discharge data are optional for model calibration. All hydrological processes are simulated within a GIS framework. Because a large part of the annual precipitation is in the form of snow, snow melt simulating is done by a model based on hourly temperature data. The conceptual temperature index or degree-day method is used in this study because of its simplicity but it has not a strong physical foundation. The method replaces the full energy balance with a term linked to air temperature. It is physically sound in the absence of shortwave radiation when much of the energy supplied to the snowpack is atmospheric long wave radiation [9, 12]. The equation is as follow:

$$M = \max[0, (K \text{ snow} + K \text{ rain } P)(Ta - To)] \tag{3.7}$$

where M implicates the daily snowmelt $[mm]$, $Ta[^{\circ}C]$ shows the mean air temperature, $To[^{\circ}C]$ shows a threshold melt temperature, Ksnow is a melt-rate factor $[md^{-1^{\circ}} \cdot C^{-1}]$, and Krain is a degree-day coefficient that shows the heat contribution from rainfall $[d-1^{\circ} \cdot C-1]$. The critical melt temperature To is often intuitively set to $0^{\circ}C$. The melt-rate factor Ks now is an effective parameter and may vary with location and characteristics of the snow. However, Ks now, To and K rain can be calibrated.

4 The study area and their characteristics

The studied area is a part of the water distribution network in Tehran, which includes 6 reservoirs that are covered by networks. The total capacity of the studied tanks is 325 thousand cubic meters. According to the customer bank information, the water and sewage company covers the number of 155,776 subscribers with a population of 1,308,923 people. The total length of the network pipes covered by these reservoirs is 119,191 km. Table 1 contains the main information of each of the reservoirs within the scope of this research.

Table 1: The main characteristics of the area under study

Reservoir ID	Capacity (1000 $m3$)	Covered Population (person)	Pipes Network Length (Km)	Stakeholders Number	Total Outflow (mcm/yr)	Area (Km^2)
A	27	85,987	110.55	7373	16,402	517
B	34	98,487	107.70	10,300	8662	1365
D	76	58,151	110.76	9014	10,781	1042
E	74	358,187	353.34	48,041	32,422	580
F	56	405,082	319.90	50,711	30,233	1262
Total	267	1,005,894	1002.25	125,439	98,500	4766

5 Findings

5.1 Water not considered in water distribution networks

In the area covered by each reservoir, the difference between the output flow from the reservoir and the consumption of the subscribers during a statistical period is not taken into account. Unaccounted for water is divided into two parts:

apparent losses and real losses. Apparent loss is the water that has been consumed, but has not been measured due to managerial, operational, personnel, measurement tool and unauthorized use errors. The real loss is the water that leaves the system through leakage from the pump houses, tanks and their overflow, the transmission line between the tanks and the distribution network. The daily average of unaccounted water during a year for each person of the city or village population is called average unaccounted water per capita. For each node, in addition to the consumption that is considered from the consumers, consumption is considered as unaccounted water. To achieve this goal, the unaccounted water during one year for the area covered by each storage tank is converted into the volume of water per second per meter of pipe length. Table 2 shows the unaccounted water per unit length of each pipe in the area covered by the six tanks.

Table 2: Unaccounted water per meter of pipe length in the range (liters per second per pipe length unit)

Water not included (liters per secondper meter $\times 10^4$)	Length of pipes (meters)	Water is notincluded (liters per second)	population range (people)	Water per capita is not taken into account (liters per day per person).	range
13.1	109.728	144	57.136	218	A
8.7	352.350	308	354.920	75	B
5.6	311.335	175	409.047	37	C
17	222.483	379	197.400	111	D
17	107.058	379	97.280	111	E
17	110.630	188	182.770	89	F
	1.213.584		1.298.553		Total

5.2 Recalibration of hydraulic model using water model is not considered

According to the hydraulic behavior of the drinking water distribution network in the area under study, in the modeling of water losses in the network (water is not accounted for), considering that studies on the exact determination of unaccounted water are not available, and on the other hand, the recalibration of the model is influenced by the amount and behavior of the water. Therefore, the difference between water production and all the measured uses was considered as unaccounted water and its different behavior in the network was investigated in relation to the behavior (pattern) of water use. The unaccounted water was divided into two parts: apparent losses (non-physical) and real losses (physical). In this study, the amount of network losses is calculated from the difference between the output of the tank (based on the recorded data of the output meter of the tanks) and the amount of water sold in the affairs of subscribers. These losses include the amount of leakage, apparent losses, meter error, unauthorized branches, etc.

5.2.1 Allocation of losses as average (annual) and without consumption pattern (fixed)

At this stage, the amount of losses calculated within the scope of each network during the year is calculated and the average amount of water flow is not calculated without taking into account the hourly changes (straight line) and is allocated to them in proportion to the length of the pipes. Due to the fact that the area of reservoir C physically had communication points (flow exchange) with other neighboring reservoirs (outside the scope of the current research) at the time of preparing the current situation model, it is possible to draw how the flow rate from the reservoir behaves with the flow rate of the current situation model. It did not exist independently for this repository. Therefore, after calculating the difference between the recorded flow pattern of the output from the reservoirs and the flow pattern of the current model in the minimum and maximum consumption values, it is possible to calculate the percentage difference between these two patterns, which is shown in table 3.

Examining the similarity percentages listed in table 3 shows that the difference between the model prepared from the existing situation and the flow rates recorded from the outlet of the reservoirs at the minimum consumption is acceptable (5 to 8 percent) and at the same time at the time of maximum consumption in the dominant number shows an unacceptable difference (16-30% for 4 out of 5 tanks with independent statistics). Therefore, the difference values at the time of maximum consumption require further investigation and improvement of the model calibration method in order to reduce the difference values to an acceptable range.

Since the consumption pattern in the water sector is not taken into account, it is one of the factors influencing the recalibration of the current situation model, and due to the lack of availability of water studies and results, the first option (not taken into account water pattern is constant equal to the average water taken into account) not arrived in the year for each hour) was considered to calibrate the model and a comparative evaluation was done with the

Table 3: The percentage of difference between the pattern of flow rate changes of the recorded values with the current situation model at minimum and maximum consumption

The difference between recorded and model discharge (%)		Tank number
maximum consumption	Minimum consumption	
30	8	A
23	7	B
16	7	D
9	5	E
20	5	F
19.6	6.4	average

recorded data of flow rate at the outlet of the tank. Since, based on scientific principles, the leakage and flow through the opening has a direct relationship with the internal pressure of the flow through the pipes, and in the drinking water distribution network, the amount of pressure in the network has an inverse relationship with consumption, so it is assumed that if the changes in the water pattern not calculated and at the same time the leakage values which are not calculated from the influencing components in the water quantities and are a direct function of the pressure in the network as an effective parameter in network recalibration in the next option of changes in the flow pattern of the current situation model in adapting to network pressure changes (According to the inverse consumption pattern in the network) will be compared and evaluated compared to the recorded data of the flow rate at the outlet of the tank.

5.2.2 Allocation of losses as an average (annual) inverse of the network consumption pattern

At this stage, the amount of losses calculated within the scope of each network throughout the year has been calculated and allocated to them on average by inverting the consumption pattern of the network subscribers (Pattern) according to the length of the pipes. After drawing the diagrams related to the behavior of the network (reservoir output flow in the hydraulic model) with the actual behavior of the output of the tank (according to the data and statistics of the output meter of the tank), it is possible to calculate the difference between the recorded flow pattern of the output from the tanks and the current model flow pattern in Minimum and maximum consumption amounts are provided.

Table 4: The percentage difference between the pattern of flow rate changes recorded values with the current situation model at minimum and maximum consumption

The difference between recorded and model discharge (%)		Tank number
maximum consumption	Minimum consumption	
14	74	A
11	36	B
12	35	D
9	17	E
9	37	F
11.4	39.8	average

Examining the similarity percentages listed in Table 4 shows that the amount of difference between the model prepared from the current situation and the flow rates recorded from the outlet of the reservoirs at the minimum consumption is from an average value of 6.4% (in the water model, it is not considered constant without changes) to an average difference. 39.8 percent (not calculated in the water model in accordance with the inverse consumption pattern) has increased, and besides that, the maximum range of difference between the model values and the flow rates recorded in the minimum consumption has increased from 8 percent to 74 percent despite the improvement in the relative average. Is. Also, the difference between the model prepared from the existing situation and the recorded flow rates from the outlet of the reservoirs at the maximum consumption from the average value of 19.6% (in the water model is not considered constant without changes) with a significant improvement to the average difference of 11.4% (In the water model, it is not considered according to the usage pattern) it has decreased with a relative improvement and while the difference between the model values and the recorded flow rates in the maximum consumption coincides with the improvement in the relative average, the difference in the maximum range has decreased from 30% to 11%. has improved. Therefore, it can be stated that the replacement of the water model that was not considered in accordance with the inversion of the consumption pattern in the recalibration of the current situation model had a significant effect on increasing the accuracy of the model at the time of maximum consumption, and at the same time, it significantly reduced the accuracy of the model at the time of minimum consumption. Is.

5.2.3 Allocation of losses on an average basis (annually) according to the network consumption pattern

In this option as well, the amount of losses calculated within the scope of each network during the year has been calculated and allocated to them on average according to the consumption pattern of the network subscribers (Pattern) according to the length of the pipes. Table 5 shows the difference between the recorded discharge pattern of the output from the reservoirs and the discharge pattern of the current model in the minimum and maximum consumption values.

Table 5: The percentage of difference between the pattern of flow rate changes of the recorded values with the current situation model at minimum and maximum consumption

The difference between recorded and model discharge (%)		Tank number
maximum consumption	Minimum consumption	
2	12	A
4	13	B
0	5	D
1	3	E
1	7	F
1.6	6	average

The model of unaccounted water (constant without changes) has improved to an average difference of 6% (in the model of unaccounted water according to the consumption pattern) and this is while the maximum range of difference between the model values and the flows recorded in the minimum consumption despite the improvement in the average Relative has increased from 8% to 13%. In addition, the difference between the existing model and the recorded flow rates from the outlet of the reservoirs at maximum consumption is from the average value of 19.6% (in the model of water not considered constant without changes) with a significant improvement to the average difference of 1.6% (in the model The difference between the model values and the recorded flows in the maximum consumption has decreased (improved) from 30% to 4% in the maximum range of the difference along with the improvement in the relative average. Therefore, the replacement of the unaccounted water model in accordance with the consumption model in the recalibration of the current situation model has had a significant effect on increasing the accuracy of the model.

5.2.4 Evaluation of predicted discharges based on the explanation factor

Three groups of discharges predicted from the hydraulic model of the current state of the network under study for calibration options based on the water pattern not considered without change (average), the reverse of the pattern of water consumption by the subscribers and according to the pattern of water consumption by the subscribers, in front of the discharge recorded at the outlet of the reservoirs, were evaluated using the statistical parameter of the coefficient of explanation, the results of which are shown in table 6.

Table 6: Evaluation of water patterns not taken into account in the recalibration of the hydraulic model of the water distribution network

Statistical parameter			The pattern of water is not considered
RMSE	MAE	R^2	
0.01	0.90	1.00	Fixed (average water not calculated per year per hour)
0.30	0.70	1.00	Reversal of expenses
0.19	0.18	0.99	According to usage

Comparing the statistical results related to the coefficient of explanation of the options of the unaccounted water model shows that for the squared error parameter ($R2$), all three options provide completely acceptable predictions. In the following, with a comprehensive evaluation and by comparing other statistical parameters such as the mean absolute value and standard deviation of the balance, it shows that the use of the inverse model option of the customers' consumption is not considered as a water model, for recalibration of the hydraulic model of the distribution network, more acceptable limits are found. In order to make the predicted values close to the recorded values of the hourly output of the reservoirs, it has been provided.

5.3 Output pressure range of the hydraulic model and measured pressures in the ranges

After determining the hydraulic behavior of the pressure nodes of the drinking water distribution network, the range was analyzed in modeling and its compatibility with the pressures measured in different areas covered by the network. It should be noted that the pressure measurement operation was carried out in the area at different times for 350 points and also the hydraulic model of the network for the day of maximum daily consumption was dynamically modeled and then the output pressures of the hydraulic model were compared with the measured pressures. Also, the number of input pressure data bank of 60 pressure relief valves in the studied network was compared with the output of the hydraulic model as points that can be checked. In total, the number of points that can be checked, including the points whose pressure is measured and the available pressure relief valves, is 410.
The output pressures of the hydraulic model are shown as pressure zones in Figure 2. In this figure, taking into account all the measured points (pressure measured points + pressure relief valve information), pressure curves have been drawn in the area of the network covered by the tanks. The coloring and pressure ranges were determined and drawn based on the following.

A- Low pressure range in the network for points with pressure less than 26 meters of water with orange color

B- Normal network pressure range for points with pressure between 26 and 50 meters of water with green color

T- Range at the threshold of high pressure for points with pressure between 50 and 60 meters of water with purple color

D- The high-pressure range of the network for points with a pressure of more than 60 meters of water in blue color

In some points of the network, due to the presence of the pressure relief valve and separation of the pressure range, two high-pressure zones (the end of the pressure zone) and low pressure (the beginning of the pressure zone) are adjacent to each other.

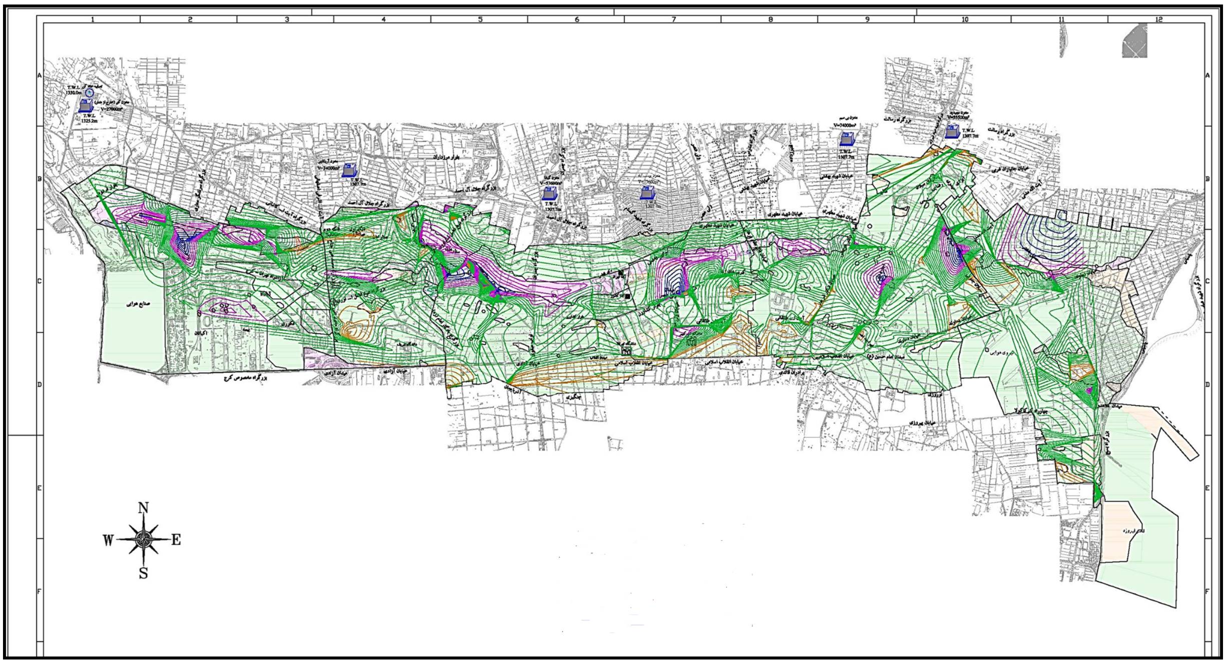

Figure 2: Output pressure zones of the hydraulic model and equal pressure lines of the measured pressure points in the range

6 Summary

The proportional distribution of flow in the nodes of the water distribution network is one of the challenges of calibration in the hydraulic model of the distribution network. In the conditions of lack of access to unaccounted water results and information, designers and engineers use the recommended experimental formulas or fixed values (average unaccounted water per year per hour) as the model of unaccounted water in the model and in an approximation They accept the results and predictions of the trust model. Due to the reduction of available resources, as well as due to overharvesting and the risks of climate change, it is necessary to estimate the amount of drinking water in the horizon of the projects (next 25 years). For this reason, the evaluation of other hypotheses as a more accurate alternative to the water values and pattern was not considered as the goal of this research, and three options were included in the evaluation process. The results of the evaluation of the three options showed that the water option was not considered for a network in accordance with the pattern of the hourly consumption of subscribers in separate networks, in terms of the percentage difference between the pattern of changes in the flow rate of the recorded values with the model of the current situation in the minimum and maximum consumption. And also based on the comparison parameters of the statistical error for the predicted values of the model with the recorded values, it provides a relative advantage and a more acceptable level of accuracy. At the same time, the accuracy of other options is relatively acceptable. Therefore, taking into account the principle of reducing leakage for increasing consumption, it has been determined that unauthorized and unregistered consumption is the dominant part (over leakage and water losses caused by incidents) in water are not counted.

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