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ORIGINAL ARTICLE

Pressure management for leakage reduction using pressure reducing valves. Case study in an Andean city

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Abstract A very common problem in distribution systems is water leakage, which can be reduced by pressure management. The objective of this study was to evaluate the reduction of water leakage by optimizing the pressure using pressure reducing valves (PRV). The corresponding hydraulic model of a real distribution network was developed using the EPANET software. After the hydraulic model was calibrated and validated, the analysis of the pressure in the nodes, the velocity in the pipes, through the technical performance indicators (TPI) was performed, in addition, the leakages were quantified. The initial results indicated the need to optimize the pressure, nodes with excessive pressures were found in the lower part of the network. WaterNetGen was used as an extension of EPANET software to model leakages based on pressure after determining the leakage coefficient and considering the installation of two PRVs. The results allowed optimizing the appropriate pressure in 30.83% of the nodes and minimizing leakages in 31.65%. In turn, the simulation assuming the installation of two PRVs determined that the TPI would increase from 79.81% to 97.45%. The focus of this study is recommended to the companies that supply drinking water as a support tool for planning to reduce leakages.

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1. Introducción

A distribution system that supplies drinking water from the points of supply to the points of consumption is made up of pipelines, valves, tanks, pumps, etc. and supplies the liquid to consumers under certain hydraulic conditions that are difficult to operate and control due to constant urban and population growth [1]. The purpose of the drinking water distribution network (DWDN) is to provide the consumer with water in the right quantity and pressure, but also in the right quality, in compliance with local regulations [2,3]. Consequently, each drinking water company must maintain high water quality throughout the distribution system [4]. DWDN are now required to be increasingly efficient to maintain water quality [5,6]. This objective is not easy to achieve due to the geometric complexity of the network, the complexity of the network connections, the different functional regulatory systems, the temporal and spatial variations in water demand and the reactions between the various substances contained in the water and the reactions between the water and the internal wall of the pipelines, as well as the existence of Non-Revenue Water (NRW) high percentages [7].

Managers of drinking water systems, worldwide, have as one of their priorities the water losses reduction, which reach 30–40% of all the water that enters the DWDN. Therefore, currently the problem of water losses is important, seeking sustainability of consumption and environmental protection [8]. To minimize water losses, pressure management techniques have been presented as a conditional parameter of the leakage indicator, for which the implementation of elements that cause pressure losses, such as pressure reducing valves (PRV) has been suggested [8,9].

Mathematical modeling of the water distribution system has been fundamental to evaluate and solve problems such as: the important leakages detection; water quality (residual chlorine); assessing the capacity of systems to meet the demands of new urban developments; and extending the coverage of water distribution service in areas above the level of service delivery through the implementation of pressure levels and pumping systems, among others [10,11]. Rossman [12] developed the EPANET model, which is a useful tool that allows to know the real-time status of the system considering current conditions and, similarly, to predict the future behavior of the city due to an ideal or non-ideal future condition. Pressure management can be achieved through the implementation of pressure reducing valves (PRV), which can be considered as an optimization problem with specific objective functions and restrictions [11]. On the other hand, the simulation of water distribution networks is an invaluable tool in the evaluation of the response of a DWDN to different operative actions or control strategies before applying the actions to a real water network [9].

Few studies has been carried out on Andean cities networks located over 2200 m.s.a.L., such as those made by [13,14]. Azogues city, located to the south of the Ecuadorian Andes has networks of pipelines for drinking water distribution that work by gravity, taking advantage of the relief and the mountainous presence. The city is located at an average height of 2450 m.s.a. L., the range of altitudinal variation of the distribution network is between 2360 and 2810 m.a.s.L. There are slopes lower than one percent and higher than one hundred percent due to the topography of the terrain, affecting the pipelines network with excessive pressures that cause leakages in the system. Therefore, it is necessary to optimize the pressures in the distribution network to avoid damage to the pipelines that produce economic losses for the public municipal company of drinking water, sewerage and environmental sanitation of the Azogues city (EMAPAL). The objective of this study was to evaluate the leakages reduction by optimizing the pressure through the implementation of pressure reducing valves (PRVs).

2. Methodology

2.1. Study area

The distribution network of drinking water on which this study was conducted is located in the Azogues city, in the south of the Ecuador Republic. It has a population of 70,064 inhabitants and an area of 1200 km^2. Fig. 1, shows a panoramic view of the Azogues city. The city's drinking water distribution network is divided into six zones. For this study an area from the Northeast was selected which corresponds to the upper area of the city, with elevations between 2479 and 2624 m.s.a.L. and a population of 5800 people approximately. This part of the network varies between 2481 and 2624 m.s.a.L. This network has been chosen because it has an updated cadaster of the distribution system, and this is where EMAPAL has detected a 46.86% of NRW [15].

2.2. Characterization of the distribution network

The distribution network for this study is of the mixed type since it has branched and meshed parts. The pipelines throughout the network are made of PVC with a length of 26.6 km. The nominal diameters of the existing pipelines are 63, 110, 160 and 200 mm with lengths of 22.31; 3.25; 0.52 and 0.56 km respectively. There are currently 80 valves, among which is a pressure reducer valve (PRV1) and the 79 are flow regulators valves (FCV). Of the 79 FCV, that are gate valves, 21 are closed, 20 are semi-open (considering that they are open between 65 and 70%) and 38 valves are fully open. The PRV1 is out of operation. The DWDN is fed from a 500 m^3 capacity reservoir, which discharges by gravity through a Ø 200 mm conductive pipeline, after which it branches out into Ø 63 mm pipelines to the furthest reaches. In this distribution network, there are 1519 sites, of which there are seven high consumption sites: three educational institutions, a church, a hospital, a stadium and a market.

2.3. Hydraulic model

The hydraulic model of the network studied was prepared following five successive stages recommended by the United Kingdom Water Research Center aimed at modeling a water distribution system: 1. construction of a network model, 2. assignment of distribution network parameters, 3 assignment of consumptions, 4. calibration of the model, 5. analysis and maintenance of the model [16]. The model was calibrated by adjusting the value of the roughness coefficient, demand in the nodes, as well as in the minor loss coefficients of the valves,

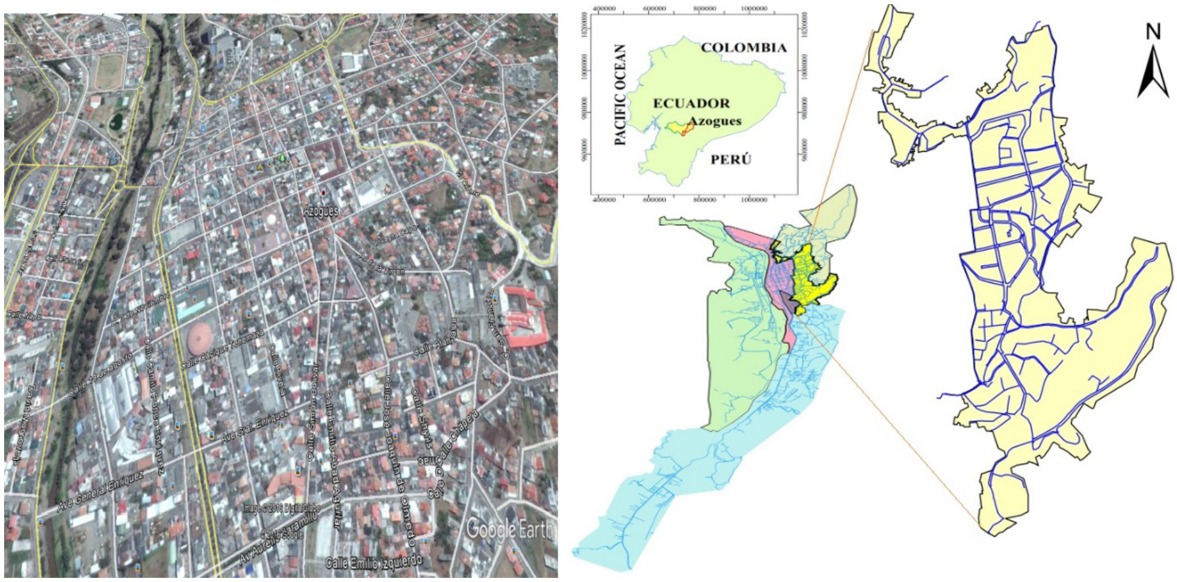


Fig. 1 Panoramic view of the Azogues city (left image), city supply system and distribution network where the study was applied (right image).

the steps were repeated until they had an affinity with the system reality [17,18].

2.4. Evaluation of the DWDN performance

To evaluate the performance and quality of the DWDNs, technical performance indexes (TPI) are used, by means of which the behavior (nodes, pipelines) of the DWDN can be evaluated, comparing it with reference values [19–20]. The TPIs present values between 0% (poor service) and 100% (efficient service). Taking into account the criteria of [19,20], the TPI_{press} was determined, considering that the nodal pressures are in the range of $p_{min} = 20$ and $p_{max} = 70$ mca, which represent adequate energy states, causing lower levels of water losses. The (TPI_{press}) can be calculated by Eqs. (1) and (2).

$$TPI_i = \begin{cases} 0, & p_i < p_{min} \\ 1, & p_{min} \le p_i \le p_{max} \\ 1 - \frac{p_i - p_{max}}{p_{max} - p_{min}}, & p_{max} < p_i \le 100 \\ 0, & p_i > 100 \end{cases} \tag{1}$$

$$TPI_{press} = \frac{\sum_{i=1}^{NN} Q_i TPI_i}{\sum_{i=1}^{NN} Q_i} \tag{2}$$

where; pi, is the nodal pressure in meters; NN, is the number of nodes in the system; Qi, is the nodal demand.

Taking into account the criteria of [20,21], the TPI_{vel} was determined, considering that the velocities are in the range of $v_{min} = 0.2$ and $v_{max} = 3$ m/s; Excessive velocities cause greater abrasion and loss of load. The TPI_{vel} can be calculated by Eqs. (3) and (4).

$0, v_i < v_{min}$

$$\frac{v_i - v_{min}}{v_{mean} - v_{min}}, v_{min} \le v_i \le v_{mean}$$

$$TPI_i = \frac{v_i - v_{max}}{v_{mean} - v_{max}}, v_{mean} \le v_i \le v_{max} \tag{3}$$

$0, v_i > v_{max}$

$$TPI_{vel} = \frac{\sum_{i=1}^{NP} Q_i TPI_i}{\sum_{i=1}^{NP} Q_i} \tag{4}$$

where v_i, is the flow velocity in m/s in pipeline I; v_{mean}, is the average velocity in pipeline i, $v_{mean} = (v_{min} + v_{max})/2$; NP, is the number of pipelines in the system; Qi, is the flow that circulates in the pipeline i.

Muranho et al. [19] also proposes the Eq. (5) to calculate TPI_{vel}.

$$TPI_{vel} = \frac{\sum_{i=1}^{NP} v_i L_i D_i^2}{\sum_{i=1}^{NP} L_i D_i^2} \tag{5}$$

where; NP, is the number of pipelines in the system; v_i, is the flow velocity in m/s; L_i and D_i, is the length and diameter of the pipeline respectively.

2.5. Evaluation and categorization of pipelines

A catalog or categorization list of all the pipelines in the distribution network was determined by means of an operation index or pipeline behavior (OI), based mainly on the results obtained in the leakages simulation in each of the pipelines, with which was intended to evaluate the consequences of pipeline failures under comparable conditions. The determination was made by adapting the methodology carried out by [22].

This index is based on two parameters: a first parameter that characterizes a pipeline and is based on the average leakage rate ($Q_{averageleakages}$) throughout the simulation. This parameter is relevant in relation to total water loss over an extended time period [22].

The second parameter is the pressure along of the simulation ($P_{average}$), this parameter affects the pipelines adjacent to the nodes where the pressure is higher than the allowed one, it is considered that the operation in those consumption points would be affected by the presence of leakagess (abnormal operation).

Each of the previous parameters that make up this index (OI) is weighted by a weight that balances the relevance of each of the parameters in the index, to the average leakage rate ($Q_{averageleakages}$) and average supply pressure ($P_{average}$) they have been assigned a weighting coefficient of 0.50 to each one (Eq. (6)).

This index is between 0 and 1, where values close to 1 will indicate a worse behavior of said pipeline in case of possible leakage or a possible emptying of the tank. Values very close to zero will indicate a good performance of the pipeline in question, since it will not generate a high average leakage rate, nor will it cause an important condition to the service [22].

$$OI = a\frac{Q_{averageleakages}}{max(Q_{averageleakages})} + b\frac{P_{average}}{max(P_{average})} \tag{6}$$

where a is 0.5 and b is 0.5.

2.6. Optimization of hydraulic networks

Water losses in the network can be minimized thanks to different techniques, including the reduction of excess pressures [23]. The process of suitable pressures optimization seeks to control the pressure ranges determined by the irregular topography of the land, locating the possible pipelines where it is feasible to install pressure regulating valves (PRV), when considering the unit power, the most feasible pipeline can be identified where the PRV can be implemented. The unitary power (UP) is the energy dissipated by a pipeline, allowing to show which are the pipelines with the greatest impact (pressures on the regulation) in the DWDN hydraulic behavior. The UP generates an added value for the network optimization process, allowing to determine the ideal locations of the PRVs [24]. Therefore, the analysis focused on the optimization of energy reduction in the system, through the installation of PRV, for which sectors that had pressures higher than those stipulated were identified. The methodology was based on determining the most optimal solution that meets the objective functions and restrictions indicated below [24].

2.6.1. Objective functions

The objective function that determines the hydraulic network behavior was the adequate pressures maximization, given by the Eq. (7) [25], which improved the nodes number with pressures within the ranges established in the Ecuadorian regulations.

$$\max f = \frac{APN}{TNN}100 \tag{7}$$

where; TNN, is the total nodes number in the supply network; APN, is the number of nodes with adequate pressure and that is defined in la Eq. (8) [26].

$$P_{min,ad} \leqslant Pi \leqslant P_{max,ad} \tag{8}$$

where; P_i, es the nodal pressure; $P_{min,ad}$ is the appropriate minimum pressure and $P_{max,d}$ is the appropriate maximum pressure.

A second objective function was the leakages total number in the pipelines of the sectors directly related to the installation of the PRV, defined in the Eq. (9) [26,27].

$$\min f = Q = \sum_{i=1}^{npl} C_L L_i P_{avr,i}^{1.18} \tag{9}$$

where; C_L, is the water leakage coefficient per unit length of the pipeline at the service pressure; L_i, is the total length of the pipeline associated with the node i and P_{avr} is the average pressure; npl is the number of pipelines that have leakages; C_L has a value of $1x10^{-7}$; for this objective function the pipelines associated to the sector where the PRVs were installed were considered [27].

2.6.2. Decision variables

The decision variables were: the optimal location of the valves, given by the installation of the PRVs on a specific pipeline; As well as the setting of the PRV, which refers to the target pressure that must be set downstream of the PRV to maintain proper service provision [28]. PRVs are instruments that are installed at strategic points in the network to minimize the amount of leakages by reducing pressure [11]. The places to install PRV were chosen by studying the network topology and its hydraulic behavior (flow and pressure) [29]. To help identify the best locations, a kind of categorization of the pipelines was carried out through an operation index of the pipelines (OI), explained in section 2.5. The VRPs were located upstream of the pipelines with high OI.

2.6.3. Restrictions

Hydraulic constraints were used according to the project purpose defined from the study area topological characteristics. The restrictions considered are: conservation of the mass in each node of the network, conservation of energy in each of the network pipelines and compliance with minimum and maximum operating pressures. The first two are guaranteed by EPANET and the third was verified with the simulation in order to guarantee the service adequate provision to the different consumers [28].

2.7. Modeling of leakages

For the pressures optimization, an optimal operating model can be used to define the pressure controls and the location of PRV, which are solved by linking a hydraulic simulation model (WaterNetGen) with a simulated annealing algorithm [29]. WaterNetGen is an EPANET extension for the automatic generation and design of synthetic water distribution network models (https://www.dec.uc.pt/~WaterNetGen/) [30]. The pressure-driven hydraulic model included in WaterNetGen allows the evaluation of leakages, is adapted to solve an optimal operating model (leakage minimization). A more detailed explanation of WaterNetGen can be found in another article [31]. The places to install PRV were chosen by studying the network topology and its hydraulic behavior (flow and pressure) [29].

The simplest approach to modeling leakages in a hydraulic model is to use Eq. (10) of flow emitters. This equation, included in the EPANET library, allows to simulate the flow of output through a nozzle or orifice by discharging into the atmosphere; therefore they can also be used to simulate a leakage in a pipeline connected to a node [24]. This equation will be used for the leakage modeling of this study.

$$Q_{Fi} = K_i(P_i)^N \tag{10}$$

where Q_{Fi} is the leakage rate at node i, K_i is the emitting coefficient at node i and that depends on the size and shape of the leakage hole, P_i is the pressure at node i, N is the leakages exponent. Germanopoulos [27] assumes that leakages are distributed uniformly along the pipeline, and proposed Eq. (9) to model existing leakages in a distribution network. The exponent N is assumed constant over the network, it is considered a theoretical value 0.5 (orifice flow), but they can reach values greater than 1, but it could vary according to the type of pipeline and leakage: 0.5 for leakages of fixed area, 1.5 for leakages whose size depended on pressure and 2.5 for longitudinal leakages, a value of 1.5 is generally considered for plastic pipelines. To simulate EPANET leakages, the leakage exponent N was defined with a value of 1.5, while the emitter coefficient was determined by Fig. 2, where i = 1,2,3....n.

The aforementioned procedure was applied in the real network to calculate the emitting coefficient of each pipeline, which was subsequently entered into EPANET for leakage modeling. The network was initially simulated in EPANET without PRV. While, once the location of the valves was determined, leakages were simulated assuming the installation of the two PRVs. In this way, it was possible to compare leakages without PRVs and leakages simulating the installation of PRVs.

3. Results and discussion

The distribution network of the upper area of the city of Azogues obtained has 362 pipelines, 387 nodes, 1 tank and 80

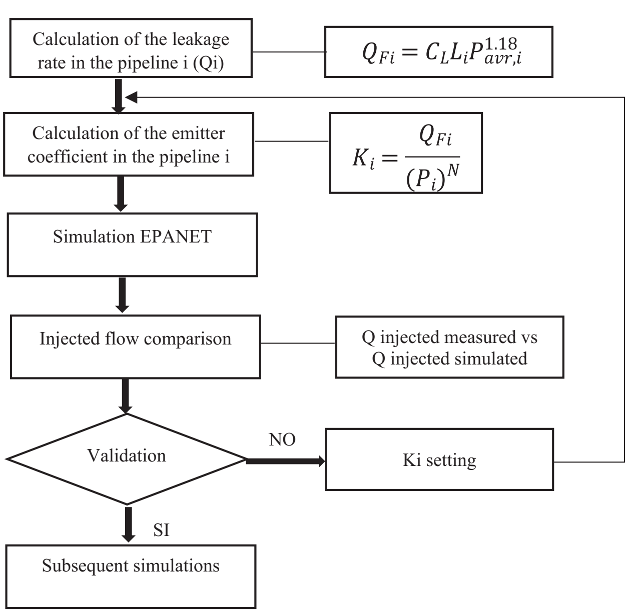


Fig. 2 Flow diagram to calculate the emission coefficients K_i.

valves, as shown in Fig. 3. It was found that the maximum demand schedule is from 8 to 9 am and the lowest consumption is from 2 to 3 am. It is worth noting the difference between night consumption (almost half of the average daily consumption) and consumption at peak or peak consumption (25% more than average consumption).

3.1. Calibration of the model

Calibration provides information on certain parameters that can be controlled during modeling but are difficult to measure (rugosity or internal diameters, or consumption modulation curves) and is performed so that the model's forecasts are as close as possible to the measurements made [17]. In this study, the traditional method of calibration (trial and error) was adopted [18].The simulated values were compared with the measured values in the field. The flow rate was measured with a portable ultrasonic flowmeter at two measuring stations that exist in the network (there are no other stations available to measure the flow rate). The pressure was measured in the households using a portable pressure gauge.

Fig. 4 shows the values of the pressure measured and simulated by EPANET, as well as the mean square error in each of the nodes, and a Pearson correlation coefficient of 0.995, which indicates a significant degree of linear dependence for a significance level of 5%, a value that is reflected in Fig. 5.

3.2. Model validation

Once the hydraulic model was calibrated, it was validated using additional data sets measured in the field under different conditions [17,18]. Validation was carried out by checking the pressure at the nodes, 22 measurements made from 8 am to 4 pm were used in the study area, these measurements were different from those used in the calibration process. In Fig. 6 the errors in each of the nodes are presented, with a Pearson correlation of 0.999 significant for a 5% of significance level; it was observed that the Root Mean Square Error (RMSE) is minimal, as can be observed in Fig. 7. As there are no significant differences, the hydraulic model represented in EPANET was validated.

3.3. Model exploration

Graphs were generated, which helped to understand and observe the results obtained. Fig. 8 shows the simulated flows at 9 am, with most of the flows in the distribution line in the upper zone being below 5 L/s, while to a lesser extent the network carries a flow greater than 10 L/s with a maximum of 27 L/s at the outlet of the distribution tank. At 2 am, which is the hour of least consumption, most of the network transport a flow of less than 5 L/s, only at the tank outlet a flow of more than 5 L/s circulates (Fig. 9).

El análisis de la velocidad del flujo en el modelo hidráulico permite concluir que existen tramos de la red de suministro de agua potable cuyo diámetro está sobredimensionada. Incluso en las horas de máxima demanda, no se alcanza en toda la red, los valores mínimos recomendados para sistemas de distribución de agua de 0.5 m/s.

In the distribution network, sediment is deposited on the bottom of the pipelines at low velocities, especially over long

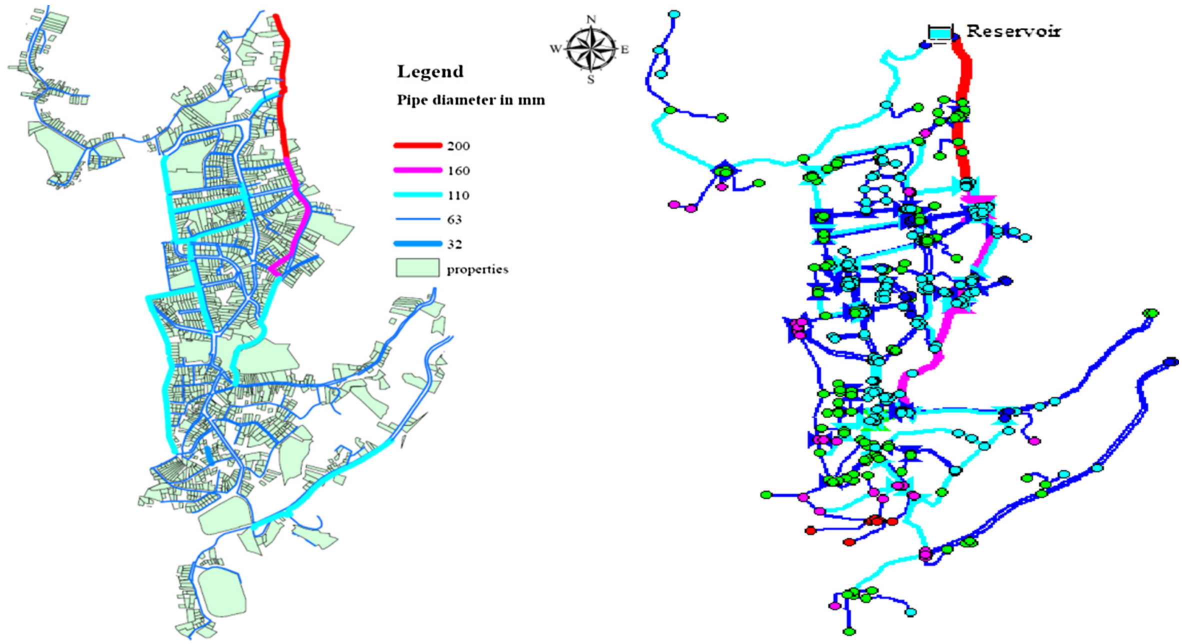
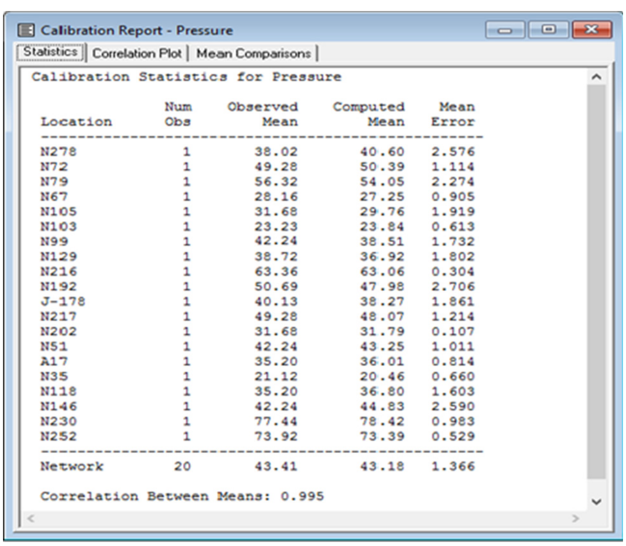


Fig. 3 (a) Distribution of the properties along the DWDN, (b) DWDN obtained from EPANET.


Calibration Report - Pressure

Statistics | Correlation Plot | Mean Comparisons

Calibration Statistics for Pressure

Location	Num Obs	Observed Mean	Computed Mean	Mean Error
N278	1	38.02	40.60	2.576
N72	1	49.28	50.39	1.114
N79	1	56.32	54.05	2.274
N67	1	28.16	27.25	0.905
N105	1	31.68	29.76	1.919
N103	1	23.23	23.84	0.613
N99	1	42.24	38.51	1.732
N129	1	38.72	36.92	1.802
N216	1	63.36	63.06	0.304
N192	1	50.69	47.98	2.706
J-178	1	40.13	38.27	1.861
N217	1	49.28	48.07	1.214
N202	1	31.68	31.79	0.107
N51	1	42.24	43.25	1.011
A17	1	35.20	36.01	0.814
N35	1	21.12	20.46	0.660
N118	1	35.20	36.80	1.603
N146	1	42.24	44.83	2.590
N230	1	77.44	78.42	0.983
N252	1	73.92	73.39	0.529
Network	20	43.41	43.18	1.366

Correlation Between Means: 0.995

Fig. 4 Calibration results considering the field measures. Date: 05.06.2018.

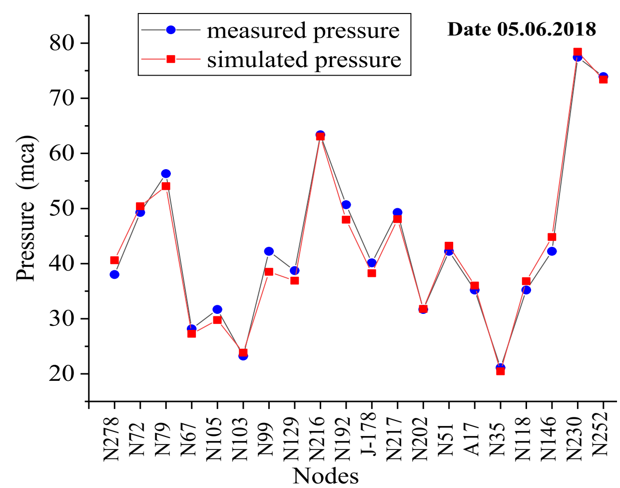
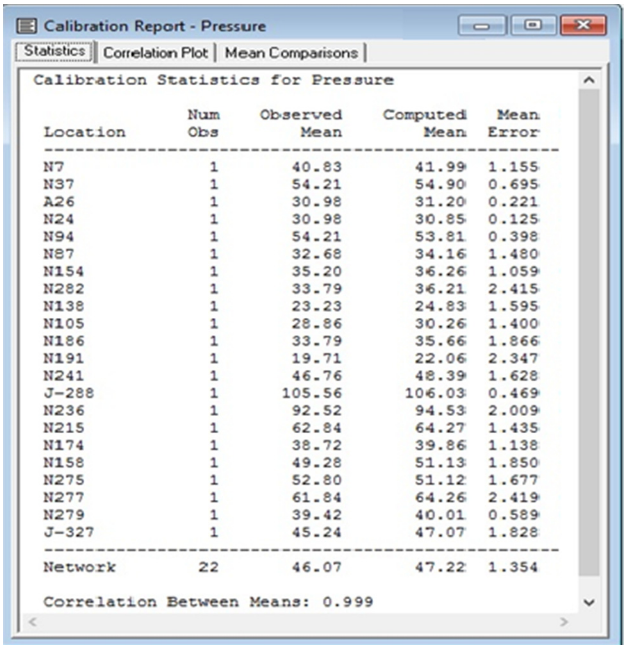


Fig. 5 Comparison of simulated pressure with those measured in the field during calibration.

distances, therefore a risk of bacterial development [32]. These low velocities, combined with the effect of chlorine in the pipelines, could promote the corrosion of domiciliary copper pipelines [6]. Therefore, according to the recommendations, the water velocity should be between 0.5 and 1.5 m/s (Ecuadorian Building Code). Sitzenfrei et al. [33] note that flow velocities in the systems vary depending on the diameter of the pipeline and range from a maximum of 2.5 m/s to approximately 0.3 m/s. Zischg et al. [20] indicate a velocity threshold of 1.5 m/s. For self-cleaning of pipelines, a velocity of 1.5 m/s is recommended for moving sediment and transporting loose deposits through fire hydrants [34]. The evaluation of the hydraulic model indicates that there are unfavorable flow velocities in the farthest sections of the network (Fig. 10, Fig. 11). At 9 am, which is the hour of maximum consumption (Fig. 12), it is observed that there are pipelines with velocity values lower than the recommended value of 0.5 m/s [35]. In many lines, at the time of lowest water demand (2 am), the conditions that cause water stagnation to prevail, the velocity is less than 0.5 m/s, even less than 0.1 m/s (Fig. 13). At the beginning of the network the highest velocities are present, at the exit of the tank the velocities are between 0.50 and 1.0 m/s, and in the farthest parts of the tank the velocity tends to decrease (Figs. 10 and 11). The analysis of the flow velocity in the hydraulic model leads to the conclusion that there are oversized sections of the drinking water supply system. Even at peak demand times, the recommended minimum values for DWDN of 0.5 m/s are not reached throughout the network.



Calibration Statistics for Pressure

Location	Num Obs	Observed Mean	Computed Mean	Mean Error
N7	1	40.83	41.99	1.155
N37	1	54.21	54.90	0.695
A26	1	30.98	31.20	0.221
N24	1	30.98	30.85	0.125
N94	1	54.21	53.81	0.398
N87	1	32.68	34.16	1.480
N154	1	35.20	36.26	1.059
N282	1	33.79	36.21	2.415
N138	1	23.23	24.83	1.595
N105	1	28.86	30.26	1.400
N186	1	33.79	35.66	1.866
N191	1	19.71	22.06	2.347
N241	1	46.76	48.39	1.628
J-288	1	105.56	106.03	0.469
N236	1	92.52	94.53	2.009
N215	1	62.84	64.27	1.435
N174	1	38.72	39.86	1.138
N158	1	49.28	51.13	1.850
N275	1	52.80	51.12	1.677
N277	1	61.84	64.26	2.419
N279	1	39.42	40.01	0.589
J-327	1	45.24	47.07	1.828
Network	22	46.07	47.22	1.354

Correlation Between Means: 0.999

Fig. 6 Validation results considering the field measures. Date: 12.06.2018.

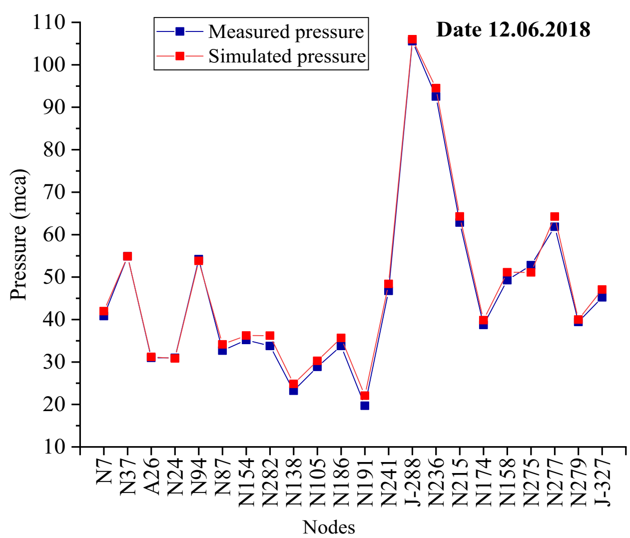


Fig. 7 Comparison of measured and simulated pressure during model validation.

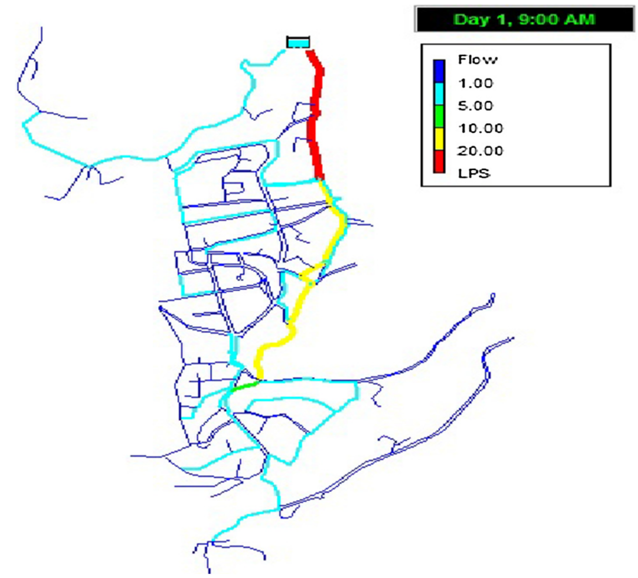


Fig. 8 Flows map (LPS) in Epanet (9 am).

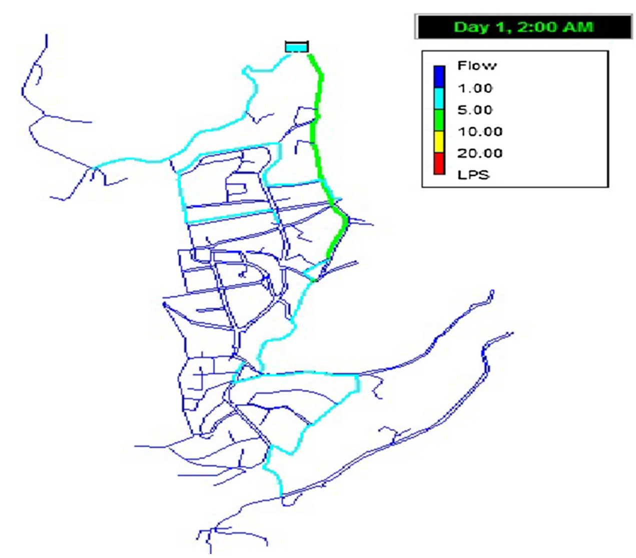


Fig. 9 Flows map (LPS) in Epanet (2 am).

The pressure is strongly linked to the orography with a lower ground level and the nodes are subject to greater pressures. According to Ecuadorian regulatory standards, the maximum pressure must be less than 70 mca. Other authors such as Aldana and López [13] recommend a pressure between 15 and 50 mca. Ramana and Sudheer [36] in their study obtained maximum pressures of 50 mca. Al-Zahrani [37] in their research reached pressures of 28–33 mca. Kepa and Stańczyk-Mazanek [35] obtained pressures of 29–54 mca. Kara et al. [38] in their study found that the optimal pressure in the network varied between 30 and 75 mca. In this study, when analyzing the distribution of pressures at the time of greatest consumption (9 a.m.), it should be noted that the highest pressures are present in the areas of greatest elevation difference (Fig. 14). At this hour, nodes with pressures greater than 50 mca were presented (higher than the values obtained by the above-mentioned authors) and some nodes even higher than 70 mca. At the time of lowest consumption (2am), several sectors are observed to have excessive pressure (Fig. 15).

The pressures are distributed in the range of 10 mca to 70 mca; with few nodes with pressures greater than 70 mca (Fig. 16) at the time of greatest demand. At the time of lowest demand there are more nodes with a maximum pressure of between 70 and 80 mca and even some nodes with a maximum pressure of more than 100 mca (Fig. 17). The excessive pressures found in this study indicate the need to install a pressure reducing valves, with the purpose of providing adequate levels of pressure and avoiding excess pressure that would cause

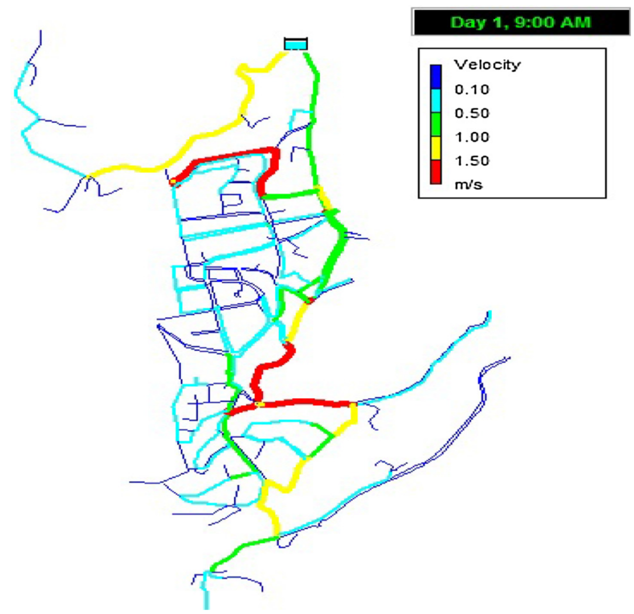


Fig. 10 Velocities map (m/s) in Epanet (9 am).

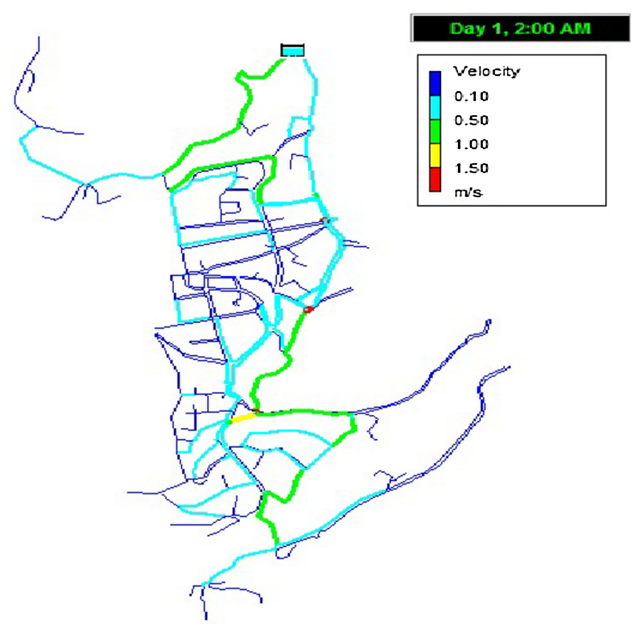


Fig. 11 Velocities map (m/s) in Epanet (2 am).

ruptures in the pipelines [11,39]. There is excessive pressure because the distribution tank is located at an elevated level with respect to the nodes. This high pressure can be related to the loss of water, which must be mitigated. Therefore, it is recommended to make an improvement and maintenance plan for the system. The highest flow rate at the peak demand hour (9am) at most nodes is less than 0.3 L/s (Fig. 16), while in the hour of minimum demand (2am) it is less than 0.1 L/s (Fig. 17).

The low velocities in the DWDN are due to the fact that it was designed to serve the city considering the expansion for a long time. When there are high pressures in the network even

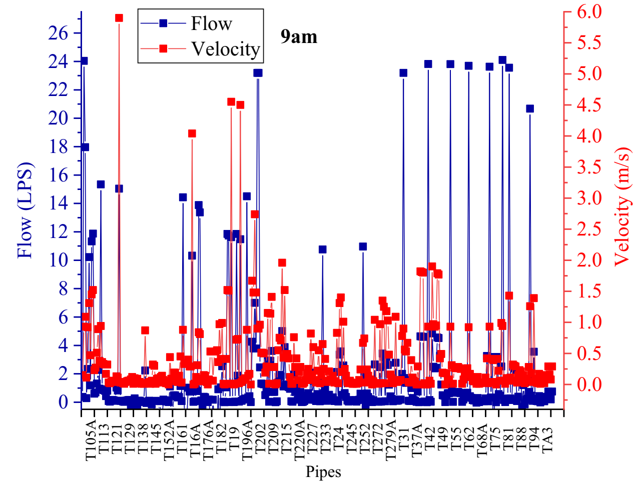


Fig. 12 Flow and Velocity in the pipelines (9 am).

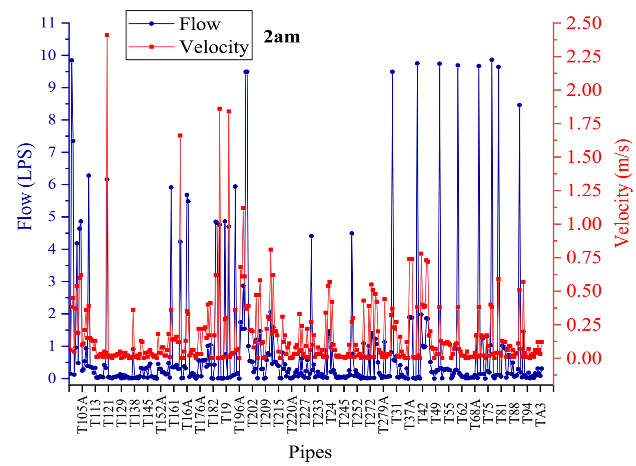


Fig. 13 Flow and Velocity in the pipelines (2 am).

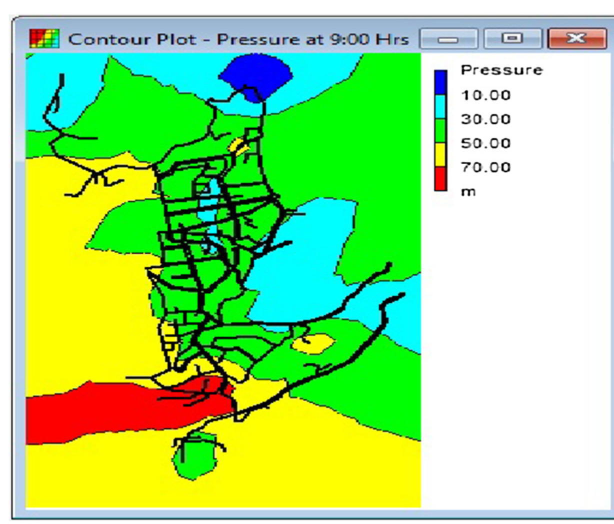


Fig. 14 Contour map - Pressures in Epanet (9 am).

higher than recommended, it ensures supply to consumers. Different theoretical approaches show that this method can be used to characterize the reliability of a DWDN [36–38].

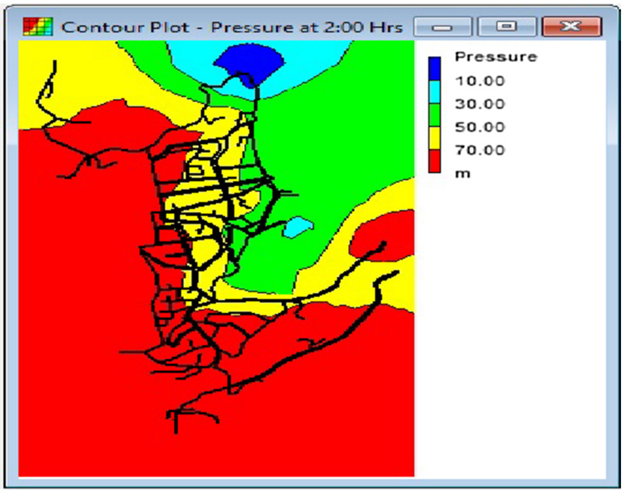


Fig. 15 Contour map - Pressures in Epanet (2 am).

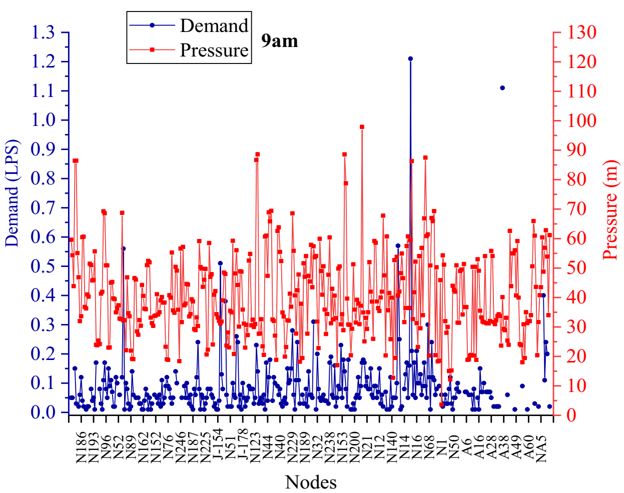


Fig. 16 Pressure and Demand in the nodes (9 am).

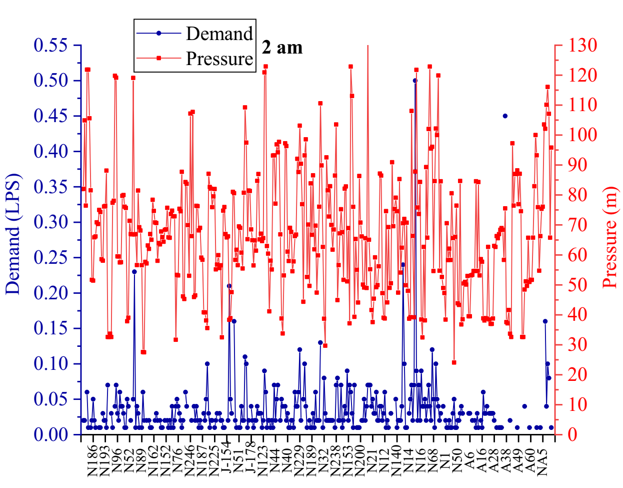


Fig. 17 Pressure and Demand in the nodes (2 am).

3.4. Performance of the DWDN

Applying the values of the pressures obtained in EPANET in the expression (2), it was determined that the DWDN has a TPI_{press} = 79.81% yield with respect to the pressure. These results reiterated the need to install pressure reducing valves to improve the TPI_{press}. Once the model was simulated, with the implementation of two PRV, as indicated in section 3.6, a TPI_{press} = 97.45% was obtained. According to these results it is recommended to EMAPAL, the installation of the PRV. On the other hand, once it was applied in the expressions (4) and (5) the velocities values obtained in EPANET, it was determined that the performance of the DWDN with respect to velocity had PTI_{vel} of 19.05% and 18.71%.respectively.

These results indicate that some pipelines are oversized, however, it is considered that these large diameter pipelines correlate with fire protection standards. With the simulation of the two PRV implementation, the TPI_{vel} calculated with expressions (4) and (5), was 20.58 and 20.37%. These low water flow velocities could reduce water quality. One way to describe the water quality is through the maximum water age, which in turn depends on the flow velocity and the length of the pipelines, the age of the water influences the growth of microorganisms. The PTI_{age} applied to the present study was calculated, using the expression recommended by Zichg et al. [20]. A PTI_{age} = 99.98% was obtained, this result indicates that the time of permanence of the water in all the pipelines is less than 24 h, only in 2 pipelines a residence time of 25 h is presented. Despite the presence of low velocities, the residence water is low, this may be due to the small length of the pipelines between the nodes, which guarantees that there is no bacterial development.

3.5. Evaluation and categorization of pipelines

The results obtained for the OI were analyzed. The first parameter (average leakage rate (L/s)) has an average leakage rate (red color) of 0.024 L/s (Fig. 18). If we order the pipelines from highest to lowest of that average leakage flow value, this value leaves below 67.21% of the pipelines that make up the network. It can also be seen how the flow rates decrease gradually from the T193 pipeline to the T259A pipeline. It can be seen that the pipelines T193, T241, T243, T244, T205, T234A show a high leakage, and these pipelines must be considered as potentially vulnerable to being replaced.

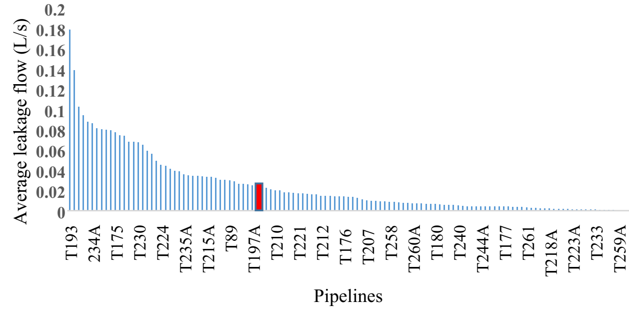


Fig. 18 Representation from highest to lowest of the average leakage rate obtained in the network pipelines.

The results obtained by the second parameter (mean pressure (mca)) are shown in Fig. 19. The average pressure value (red in the figure) is 67.54 mca. If we order the pipelines from highest to lowest according to said average pressure value, this value leaves below 57.02% of the pipelines that make up the network. It can also be seen how the results obtained are decreasing from the T234A pipeline to the T192 pipeline. It can be seen that the pipelines T234A, T235B, T233, T230B, T232 potentially vulnerable to rupture events due to their high pressures.

High pressures affect the pipeliness adjacent to the nodes where the pressure is higher than allowed, with a high potential for the presence of leakages (abnormal operation). Pressure management can reduce pipeline leakages, as well as avoid potential ruptures. The OI is dimensionless and theoretically comprises from 0 to 1; being the values close to 1 those that reflect a worse behavior in the network. The average value of this index (red color in the graph) is 0.38 (Fig. 20). If we order the pipelines from highest to lowest in that index, this average value leaves below to the 54.1% of the pipelines that make up the network. In this case, about 12 pipelines are the ones that stand out from the rest, with an OI > 0.7, from this value it can be considered relevant, determining that management on these pipelines should be carried out mainly.

Those pipelines that could present major problems are located in the lower part of the network. The pipelines T234, T244, T234, T241, T2590 and T193 have high OI values, mainly due to the difference in energy with respect to the tank.

3.6. Optimization of network operation

According to the results presented in the previous paragraphs, the need to implement two PRVs was established, since the sector has independent zones in the lower part of the network, the installation of a pressure reducing valve (PRV2) of 2″ was determined on the T198 pipeline, near the N211 node; the other valve (PRV3) was located in the T199 pipeline, near the N200 node. The location of the valves is shown in Fig. 21. PRVs maintain preset downstream pressure regardless of upstream pressure. The location of PRV was determined to be in pipelines upstream of the pipelines that presented high OI, in addition to using the reference pressure technique for the location of valves recommended by (Gupta et al. 2017) [40].

Regarding the objective function of maximizing adequate pressures; at the critical time (2am), it was determined that the hydraulic sector maximized the appropriate pressures

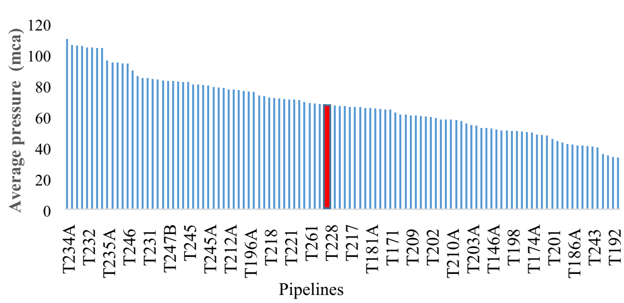


Fig. 19 Representation from highest to lowest of the average pressure obtained during simulation.

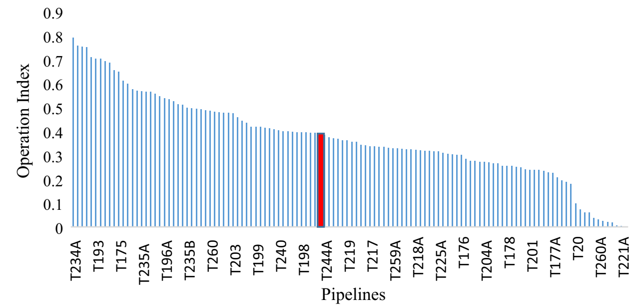


Fig. 20 Representation from highest to lowest of the OI obtained in the network pipelines.

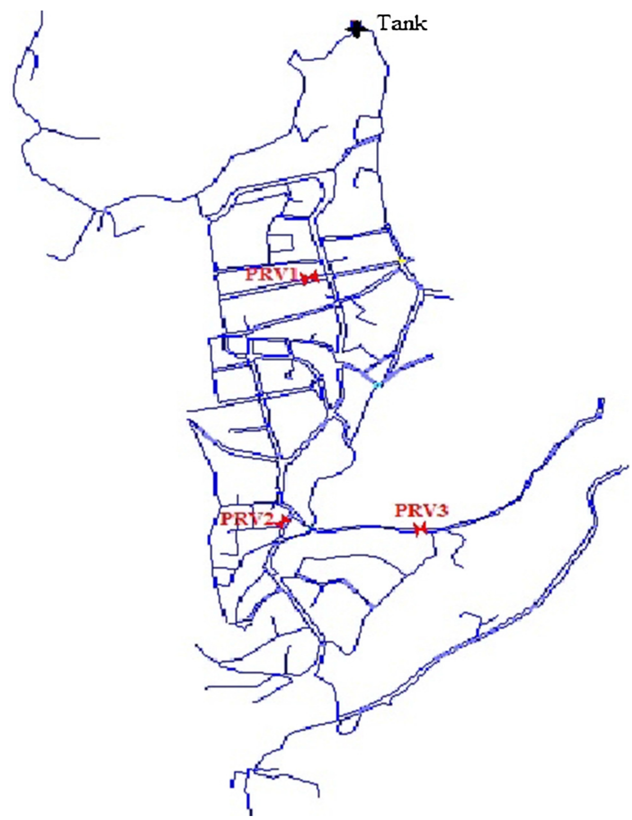


Fig. 21 Location of the new valves (PRV2 and PRV3).

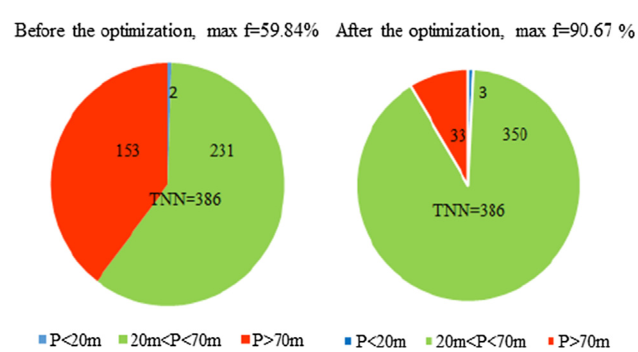


Fig. 22 Maximization of adequate pressures coverage. Before and after the optimization.

according to the specific ranges as shown in Fig. 22, taking into account that have a 59.84% of appropriate pressures (before installing the PRV) to achieve a 90.67% of appropriate pressures (after installing the PRV), which indicates an increase of more than 30% of them, considering an efficient measure to improve the behavior of the distribution network (Fig. 23).

The pressure contours are presented in Fig. 23: (a) before installing the two PRVs (2 am), 62% can be observed with pressures greater than 80 mca; (b) after installing a new PRV2 valve (2 am), 22% can be observed with pressures greater than 80 mca; (c) after installing the two new valves to the PRV2 and PRV3 system (2 am), 11% can be observed with pressures greater than 80 mca; (d) before installing the two PRVs (9 am), 21% can be observed with pressures greater than 60 mca; (e) after installing a new PRV2 valve (9 am), 18% can be observed with pressures greater than 60 mca; (f) After installing the two new valves to the PRV2 and PRV3 system (9 am), 10% can be observed with pressures greater than 60 mca. At 9 am, there is no considerable decrease in pressure when installing a new valve (PRV2), while if a second valve (VRP3) is installed, there is a considerable decrease in pressure.

According to the topography of the sector, there are several defined pressure ranges, so between 2624 m.a.s.L. (location of the distribution tank) and 2560 m.a.s.L. the pressure varies between 20 and 70 mca, in this unevenness of 64 m, the slope of the land is 4.52%. In as much, that from the 2560 and the 2481 m.a.s.L. (node of smaller height of the network) the pressure is greater to 70 m, therefore, it is defined as critical height the 2560 m.a.s.L., where the pressure starts to be greater than 70 mca; in this difference of 79 m, the slope of the terrain is 16.7%. It can be noted that the sudden variations in slope influence the distribution network hydraulic behavior.

Regarding the objective function of leakage minimization, at the critical time (2 am), according to the simulation, it was determined that the hydraulic sector would minimize the leakage with the installation of the two PRVs. Taking into account that it would go from having losses due to leakages of 381.59 m^3/d (before installing the PRVs) to 260.79 m^3/d (after installing the PRVs), managing to reduce losses of 120.8 m^3/d (31.65% of leakages), which confirms that the implementation of the PRVs it is an efficient measure that would improve the distribution network behavior.

The aforementioned values were calculated using Eq. (9), where: C_L is the leakage coefficient per unit length of the pipeline at the service pressure (1×10^{-7}); Li is the total length of the pipeline associated with node i and Pavr is the average pressure (measured before installing the PRVs and after installing PRVs), npl is the number of pipelines that are leaking [27]. The rate of 381.59 m^3/d was calculated using pressure values measured in real conditions (without PRVs); meanwhile, 260.79 m^3/d was calculated considering the pressures that would be obtained if the PRVs will be implemented.

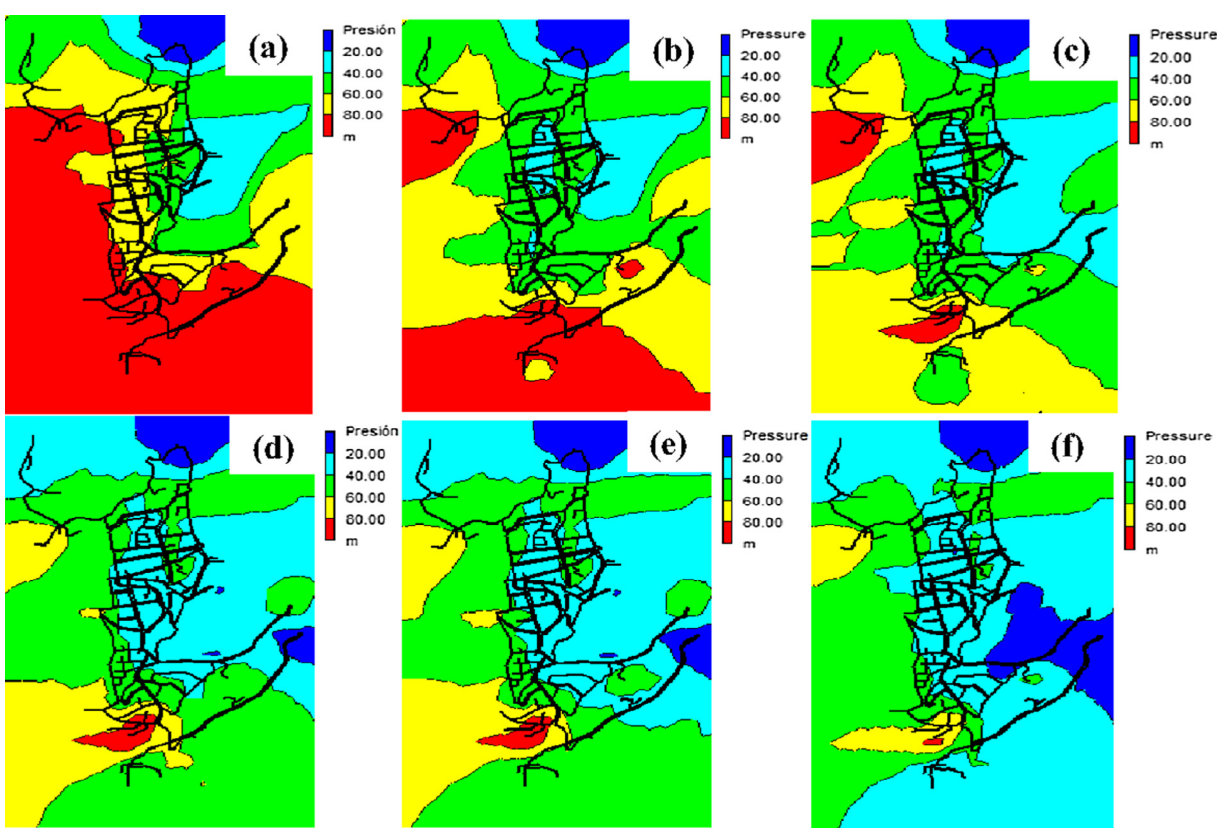


Fig. 23 Pressure contours in WDN. (a) before installing the new PRVs (2 am), (b) after installing the PRV2 (2 am), (c) after installing the PRV2 y PRV3 (2 am); (d) before installing the new PRVs (9 am), (e) after installing the PRV2 (9 am), (f) after installing the PRV2 y PRV3 (9 am).

3.7. Leakages modeling

The result of this procedure was the installation of two PRVs in those sectors that presented pipelines and nodes with excessive leakage volumes and high pressures. The simulation of leakages with WaterNetGen (EPANET extension) assuming the installation of two PRVs are presented in Fig. 24, where the evolution of leakages in the system nodes can be observed. In reference to the simulation in dynamic state, it was performed for a total of 72 h.

Fig. 24 shows the leakage contours: (a) before installing the two PRVs (2 am), the highest leakage rate can be observed with 4.41 L/s; (b) after installing a new PRV2 valve (2 am), a leakage rate of 3.32 L/s can be observed; (c) after installing the two new valves to the PRV2 and PRV3 system (2 am), minor leakages can be observed with a rate of 3.02 L/s; (d) before installing the two PRVs (9 am), a leakage rate of 2.25 L/s can be observed; (e) after installing a new PRV2 valve (9 am), a leakage rate of 2.12 L/s can be observed; (f) after installing the two new valves to the PRV2 and PRV3 system (9 am), the lowest leakage rate of 2.04 L/s occurred. In Table 1, it can be seen that the greatest decrease in leakages at 2 am would be 25.67% if PRV2 would be implemented, while if PRV3 would be implemented the decrease would be 31.65%, that is, there is an improvement in 5.98% when implementing a second valve in the distribution network. There is no considerable decrease in leakages at 9 am, with a decrease of 7.62% with the PRV2 and a 9.48% when implementing a second PRV3 valve. Table 1 shows the leakage rate and the percentage of leakage reduction for other simulated hours calculated with Eq. (9).

Table 1 shows that the leakage reduction rate was lower when it was simulated when implementing two PRVs. The reduction of leakages in terms of the percentage obtained after using two PRVs was an average of 24.03%, which is more favorable if only one valve will be implemented. A considerable reduction in pressure was observed after placing the PRVs.

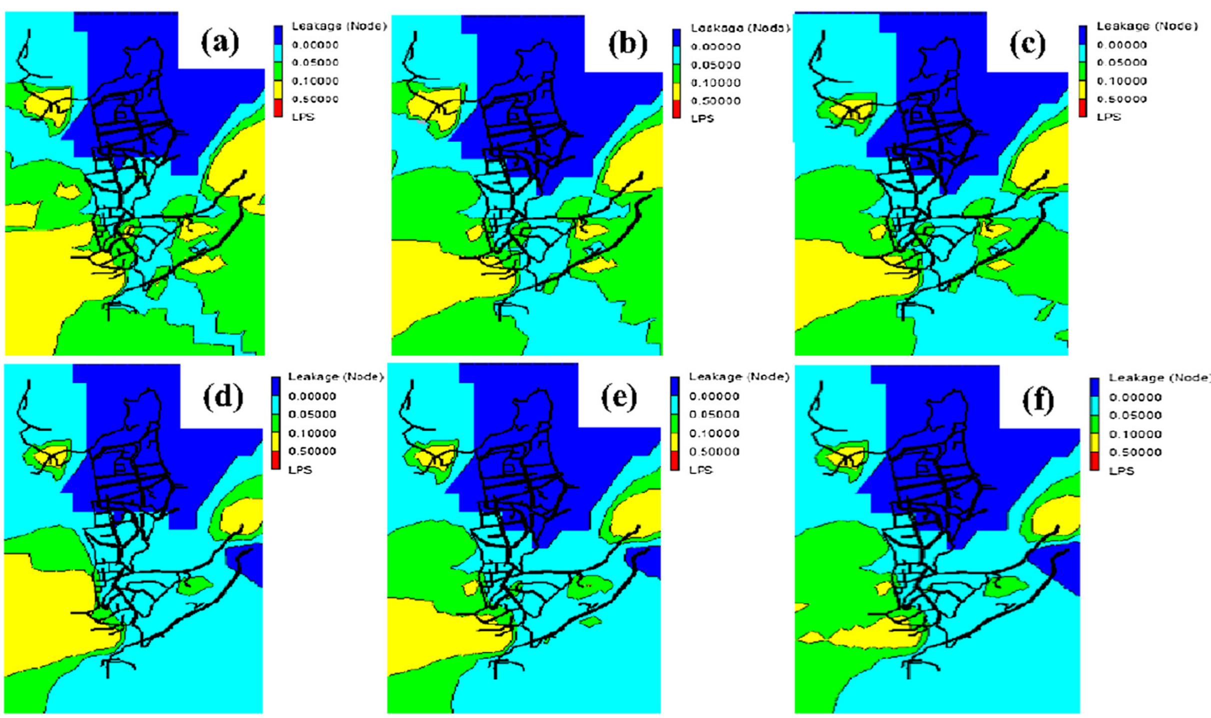


Fig. 24 Leakage contours in DWDN. (a) before installing the new PRVs (2 am), (b) after installing the PRV2 (2 am), (c) after installing the PRV2 y PRV3 (2 am); (d) before installing the new PRVs (9 am), (e) after installing the PRV2 (9 am), (f) after installing the PRV2 y PRV3 (9 am).

Table 1 Leakage rate (L/s) and leakage percentages in different time instances.

Daily time	Leakage rate (L/s)			Leakage reduction (%)	
	Without PRV	With PVR2	With PVR3	With PVR2	With PVR3
2:00 a. m.	4.41	3.28	3.02	25.67	31.65
5:00 a. m.	4.09	3.09	2.83	24.38	30.74
9:00 a. m.	2.25	2.08	2.04	7.62	9.48
12:00 m	2.76	2.31	2.15	16.25	21.88
8:00p. m.	3.15	2.44	2.32	22.59	26.39
Average	3.33	2.64	2.47	19.30	24.03

It should be emphasized that the simulations performed in this study were made assuming the installation of the PRVs. Reason why the EMAPAL is recommended to install these two PRVs.

The results allow us to verify that the drinking water distribution system can be improved, considering as a basis for studies by optimizing the pressures to reduce leakages.

With this study, it is ratified that the management of drinking water operating companies is based on identifying the main shortcomings in the operation of the networks, seeking to reduce water losses and operating costs, which will optimize the water index not accounted for in a short time.

4. Conclusions

The efficiency and performance of the actual distribution system were simulated with EPANET. Excessive pressures were found in the lower part of the network, suggesting the need to install two PRVs, thereby maximizing adequate pressures, increasing from 59.84% to 90.67% of nodes with admissible pressures.

Before simulating the installation of PRV, there was a TPIpress = 79.81% and a TPIvel = 19.05%. Once it was simulated assuming the implementation of two PRVs, the TPIpress = 97.45% increased, while there was a slight increase TPIvel = 20.58%. A PTIage = 99.98% was obtained, this result indicates that the time of permanence of the water in all the pipelines is short, which would guarantee that there is no bacterial development.

Leakage reduction through pressure optimization, using pressure reduction valves, is an effective way to improve the performance of a water distribution system (DWDN).

The pipeline operation index (OI) allowed to categorize and identify the pipelines in a simple way, taking into account the effects of a hypothetical fissure that could exist in the network. The results have been represented as a support tool for the prioritization of investments in maintenance, the need to install PRVs and possible renewal of pipelines.

The EPANET software using the WaterNetGen extension was used to simulate leakages, after determining the emitter coefficient and location of the PRVs. The leakage volume would decrease at the time of maximum pressure by 31.65% if the pressure will be optimized by installing two PRVs.

The methodology performed offers improved system performance, showing successful results for small and medium water networks.

Declaration of Competing Interest

Author declares that there is no conflicts of interest.

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