Martínez-Solano, FJ.; Iglesias Rey, PL.; Mora Melia, D.; Ribelles-Aguilar, J. (2018). Combining Skeletonization, Setpoint Curves, and Heuristic Algorithms to Define District Metering Areas in the Battle of Water Networks District Metering Areas. Journal of Water Resources Planning and Management. 144(6):04018023-1-04018023-7. https://doi.org/10.1061/(ASCE)WR.1943-5452.0000938

COMBINING SKELETONIZATION, SETPOINT CURVES AND HEURISTIC ALGORITHMS TO DEFINE DISTRICT METERING AREAS IN THE BATTLE OF WATER NETWORKS DISTRICT METERING AREAS

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ABSTRACT (150 – 175 WORDS)

The problem presented in this edition of the Battle of the Water Networks is to define District Metering Areas (DMAs) in a large network. The problem was faced in two phases. First, the complexity of the network was simplified by dividing it into three operational areas. Second, an optimization algorithm defined DMAs, looking for the best feasible solution. A preliminary simulation of the network was made. From it, engineering judgment allowed for defining of an initial set of elements suitable to change. In the second stage, a heuristic algorithm was used to search for the best DMA definition by selecting the location and setting of the pressure reducing valves and isolation valves. The network was then divided into two categories: the main pipes and the distribution pipes. Only the distribution pipes can be closed. With these restrictions and the ones described in the problem, the algorithm looks for the best DMA definition based on both the pressure and demand distribution among all the DMAs.

Keywords: Setpoint curve, Optimization, Analysis, Skeletonization

INTRODUCTION

The technical management of water distribution networks (WDNs) requires the use of increasingly sophisticated and computationally efficient applications. In this regard, it is helpful to have a calibrated mathematical model of the network and some methodologies for network optimization. This optimization may involve tasks such as the sizing of new pipelines to replace existing ones, the installation of new pipes in parallel with existing ones, the selection of new pumping units, the enlargement or installation of regulation tanks, the defining of District Metering Areas (DMAs), and the installation of pressure reducing valves (PRVs) at the entrance of every DMA to control the pressure and reduce the volume of water lost.

All these actions have a main goal: to provide for the demand at an acceptable level of service. This main goal can be divided into some specific ones. On the one hand, the consumed volume must not exceed the production capacity of the water treatment plants (WTPs). Furthermore, demands should be serviced at a suitable pressure. Finally, the supplied water must have a certain level of quality. All these goals must be achieved by minimizing the necessary investment, the energy consumed by pumping groups and the number of operations and actions over the network.

During previous editions of the Battle of the Water Networks, different algorithms have been developed to optimize the pumping rate (Marchi et al. 2014) or to reduce the background leakage (Giustolisi et al. 2016). In one way or another, all these algorithms have used the concept of DMAs as a tool for controlling the network.

Nowadays, it is very common for engineers to have to improve the hydraulic conditions of networks in service for different reasons: they were not designed optimally, the population has increased or the network has lost its efficiency. One option for improving the conditions of the WDN consists of managing the pressure on the network by installing PRVs at strategic points or dividing the network into independent hydraulic sectors or DMAs. The use of DMAs helps to reduce leakage losses in the network and extends the life of the pipes. However, to manage the network efficiently, it is necessary that these DMAs are properly sized.

The problem of DMA definition in water networks is not new (Kernighan and Lin 1970), and most of the literature has been focused on the issue of leakage reduction and the optimal localization and setting of PRVs. In this sense, different models have been proposed based on both traditional methodologies and meta-heuristic approaches (Araujo et al. 2006; Liberatore and Sechi 2009; Vairavamoorthy and Lumbers 1998). In a certain way, the segmentation of any system (electric, water supply, ...) basically involves the consideration of the network as a graph. This graph is recursively subdivided into a number of sectors or areas that meet a series of objectives. For example, Tzatchkov et al. (2008) apply graph theory to identify independent hydraulic zones, basing the definition of DMAs in the coverage area on supply sources. Subsequently, Di Nardo et al. (2014) used graph theory to define DMAs, taking as a reference energy efficiency rates. Additionally, some authors have proposed hybrid approaches combining graph theory with meta-heuristic algorithms to find an automatic partitioning of a WDN (Di Nardo and Di Natale 2011).

At the Water Distribution System Analysis Conference (WDSA) held in Cartagena (Colombia) in July 2016, a new challenge was proposed. This problem, called the Battle of Water Networks District Meter Area (BWNDMA), deals with a large water distribution network called E-Town that should be divided into DMAs in a way that some restrictions were guaranteed. A full description can be consulted in BWNDMA Committee (2016). The problem was faced using multistage approaches by other participants (Gilbert et al. 2017; Salomons et al. 2017). In this paper, the problem was solved combining engineering judgment and heuristic optimization depending on the nature of the section being analyzed.

PROBLEM DESCRIPTION

The network of E-Town has almost 14000 pipes and more than 11000 nodes. The total length of pipes is 873 km and the diameters rank from 55.7 mm to 1524 mm. An average demand of almost 2 m^3/s is supplied from three WTPs and two pumping stations. The main objective of the problem consists of defining an optimal partition which accomplished a series of restrictions described in the problem statement (BWNDMA Committee 2016). These restrictions include maximum and minimum pressures, minimum number of DMAs, limited number of entrances to DMAs and limits in the tank levels. The solution is ranked based on different objectives also described in BWNDMA Committee (2016).

The water network of E-Town might be divided into three major areas: North, South and Center. This division is based on the supplying WTPs as can be seen in Figure 1. During dry season pumps might be used with limited capacity and during wet season pumps must be switched off. The main data from each operational area after the partition are collected in Table 1. Up to 55 demand nodes might be outside the DMAs and are labeled as Municipality nodes.

METHODOLOGY

Before applying any methodology for solving the BWNDMA problem, it was necessary to conduct a preliminary study of its behavior. First, a mass balance study was done in order to identify potential supply problems. Then, an analysis of the desired supply configuration for the main pipes was also studied in order to check system deficiencies. Finally, the strategy to solve the problem was adopted.

Analysis of the water balance

The first observed problem at E-Town was the water scarcity. The maximum flow per water source is limited depending on the season of the year. For the rainy season, the maximum available flow for the whole network is 2850 l/s, while for the dry season it is 2080 l/s. Furthermore, the maximum flows at the Mohan and Fagua pumping stations are 206 l/s and 314 l/s, respectively. Fagua is directly connected to tank 12 and due to its elevation the flow is limited to 260 l/s. To deliver 200 l/s, Mohan pump curve should supply flow to nodes with an elevation under 30 m. However, in that part of the city there are some nodes with higher elevation that need to be supply from other sources (tank 15). Then, it can be seen that the dry season is the most restrictive scenario. For this reason, most of the studies were done with restrictions corresponding to the dry season scenario.

According to the model, the average demand in the network is 1924.5 l/s. It must be taken into account that the pattern average is 1.00375, with the extreme values of 0.53 at 2 am and 1.54 at 7 am. This leads to a peak demand of 2956.6 l/s. Comparing these values with the maximum allowable flow from the sources during the dry season, it can be concluded that the network cannot supply the demand directly. Furthermore, the margin between the maximum flow supplied (2080 l/s) and the base demand flow (1924.5 l/s) is only 155.5 l/s. Bearing in mind that the demand is 1.54 times its average value at peak hours, only an average demand of 101 l/s can be supplied directly from the main pipes without using tanks. So, a first conclusion has arisen: water sources must supply flow directly to tanks, and pumps must be able to supply their maximum flow. Otherwise, the flow at the peak hour cannot be supplied. Additionally, as a result of the use of tanks, the water age is expected to rise. Since there are several criteria for the design of the supply strategy for this system, water quality issues were set aside during the optimization process and were adjusted at the end of the process using Best Management Practices (BMP) applied to tanks (Gauthier et al. 2000) and transport pipes (Farmani et al. 2007).

Definition of operational areas

The size of the network implies the need for maximum simplification of the problem in order to limit the computational time. In addition, as a conclusion of the analysis of the water balance, the water sources should supply the tanks and the latter supply the demand to absorb the time variation. Finally, the water utility proposed dividing the system into three main operational areas (BWNDMA Committee 2016). The first one is supplied by the Bochica WTP in the north. The second one includes the biggest part of the system in the center that is supplied by the Cuza WTP through tanks 1, 12, 13 and 16. Finally, the Bachue WTP should supply the south part, partially aided by the Cuza WTP using tanks 14 and 15. As during the dry season the water coming from WTPs is insufficient, the Fagua and Mohan Pumping Stations help in the supply of the center and south networks. However, this configuration presents limitations during the dry season. All these facts reduced considerably the number of feasible solutions. Hence, a closer look at the desired supply configuration was done.

As part of the definition of those areas some tanks were decommissioned. Tank 2 has an elevation of 65 m and supplies demand nodes with elevations of 60 and higher. In this case the tank will not be able to satisfy the minimum pressure restriction. The same conclusion was reached with tanks 8 and 9. Elevation of tank 13 is 61 m. However, there are nodes with elevation over 40 connected to it through long pipes with small diameter. The head losses make difficult to accomplish with the pressure restrictions.

The main data from each operational area are collected in Table 1.

Overall description of the methodology

As a result of the preliminary study, two conclusions were reached. First, the sectorization strategy depended on the complexity and hydraulic viability. Second, it was observed that the hydraulic calculation took a long time. There was a need for reducing computation time. The Figure 2 shows the methodology followed for solving the BWNDMA.

The definition of the DMAs was conditioned for the hydraulic feasibility of the solutions. Since the pressure within a DMA must rank between 15 and 60 m, as a first approach, a new restriction was assumed: nodes with a difference in elevation larger than 45 m must belong to different DMAs. Additionally, in some parts of the network some DMAs are topologically defined. That is the case of nodes downstream tanks 3 and 4. So, the areas having important differences in node elevations or little transport capacity in pipes were solved using engineering judgement. This was the case for the North I, North II and the South II areas. No automatic sectorization mechanism could guarantee both pressure restrictions.

On the other hand, when there were no topographic restrictions, a combined algorithm was used that evaluates the clustering alternatives provided by METIS (Karypis and Kumar 1998b) using a Pseudo-Genetic Algorithm, PGA (Mora-Melia et al. 2013). This was the case for South I and all the Center areas.

In order to reduce computational times, the first simplification applied to the network consisted of using a scenario simulation instead of a 168 hour long extended period simulation. Three scenarios were selected: maximum, minimum and average demands. These cases covered the situations where the pressure is minimum and maximum and allow for the evaluation of the tank behavior. The maximum demand occurs at 7:00 am, and the pattern value is 1.54. On the other hand, the minimum demand corresponds to 2:00 am, with a demand pattern value of 0.53. With this simplification, it was possible to pass from 168 time steps to only three, speeding up the simulation. Furthermore, it allowed contrasting extreme values for both flow and pressure.

Engineering judgment DMA configuration.

The main difficulties in defining the DMAs are the constraints of minimum and maximum pressure. In addition, nodes with large elevation differences can not belong to the same sector. This was the case of the North area (North I and North II) and the South I subarea. These main areas have been segmented using a system based on uniform criteria dimensions, trying at all times to maximize the flow in each DMA. The process for configuring these networks was done using engineering judgement. Next, a detailed explanation of this analysis is given.

As average demand flow is slightly larger than the maximum allowable flow during the dry season, it is advisable to use as much tank capacity as possible. Water must be stored during low demand hours to supply peak demands. In other words, tanks should supply as much water as they can, somehow fixing the DMA size. So, an analysis of the tank capacities was done.

To study the capacity of each of the tanks, a standard configuration thereof was considered. This configuration (shown in Figure 3) consists of an entry controlled by a flow control valve

(FCV) allowing inflow Q_{in} and a variable output flow Q_{out}, according to the defined demand pattern $C_m(t)$. This demand pattern was applied to the accumulated base demands of all the nodes fed from the tank (Q_m).

To ensure correct operation of the tanks, the inlet flow Q_{in} must equal the average demand flow. Since the average demand pattern is different from unity, the relation between Q_{in} and Q_m can be written as:

$$Q_{in} = \lambda \cdot Q_m \tag{1}$$

with λ being the average value of the demand pattern, that is, $\lambda = 1.00375$

Next, a parametric analysis of the tank behavior was performed. This analysis was based on the flow balance in the tank, so that the net inflow equals the net outflow. If a constant inflow is assumed and the outflow is defined by the demand pattern, the moment when the tank level reaches its maximum level restriction (90% of the total capacity) happened 4 hours after the simulation begins. Repeating the same calculation for the lowest level, this occurred 20 hours after the start (as shown in Figure 4). However, in the latter case, the level reached is 49%. Since the minimum level is 10% of the total capacity, the situation corresponding to hour 4 is considered to be the worst one.

In that moment (hour 4), the volume in the tank is calculated with the expression:

$$\mathrm{V}(4) = \mathrm{V}(0) + \left(5\lambda - \sum_{i=0}^{4} \mathrm{Cm}_i\right) \cdot 3.6 \cdot \mathrm{Q}_m \tag{2}$$

Since the maximum volume at hour 4 is 90% and the volume is initially 60% of the tank volume (V_D), the maximum demand Q_m to be drawn from the tank can be obtained. The value for the demand $Q_{m,max}$ is given by the expression:

$$Q_{s,max} = \frac{0.3 \cdot V_D}{3.6 \cdot \left(5 \cdot \lambda - \sum_{i=0}^{4} C_m\right)} = \frac{V_D}{12 \cdot \left(5 \cdot \lambda - \sum_{i=0}^{4} C_m\right)} \quad (3)$$

Given the values of the demand pattern:

$$\sum_{i=0}^{4} C_m = 3.28 \quad (4)$$

then the value of $Q_{m,max}$ is given by:

$$Q_{s,max} = \frac{V_D}{60 \cdot \lambda - 39.36} \quad (5)$$

This study determines the maximum flow that each of the configured tanks is able to control according to the scheme in Figure 3. This maximum flow rate for each deposit is shown in Table 2. In this table, the maximum flow that each tank can supply is based on the expansion that will be performed. The values of these expansions were defined in the problem description of the BWNDMA.

To sum up, the consideration of whether or not to extend each tank is established based on the flow Q_m stocked. Therefore, it is a value which can be defined once a DMA definition for the network is made. In some cases, it can be stated that the tank must be enlarged. For example, the system formed by tanks 3 and 4 has a storage capacity of 470 m^3 (summing capacity of both tanks), which leads to a maximum demand of 22.6 l/s. However, these tanks must supply a total average demand of 37.39 l/s. As a result, one of the tanks (tank 3 in this case) must increase its storage capacity. This analysis was done for every tank once the DMAs were defined.

Apart from the elevation problems, South II presented an additional problem. Some of the highest nodes were supplied by a single pipe crossing a low elevation area. In addition, such high nodes could not be supplied by the pumps in the Mohan Pumping Station nor tank 14 since their maximum heads were not high enough. This considerably reduced the space of solutions. In fact, flow supplied from the Cuza WTP must exceed the maximum allowed flow.

Therefore, two decisions were made. First, the Bachue WTP would not be able to supply its maximum flow, since it supplies tank 14. Tank 15, fed by the Cuza WTP, was used instead. The second decision referred to the number of DMAs. The criteria based on the number of DMAs (15 DMAs were desirable) was neglected. The uniformity of the DMAs (BWDMA Committee, 2016) was assumed instead. The main criteria in these cases consisted of ensuring minimum pressure in the highest nodes at peak hour (07:00) and of avoiding excessive pressure in low nodes at the lowest demand hour (02:00).

The optimization process

The Center area and the South II subarea have been jointly segmented using the METIS generation algorithm weighted graph (Karypis and Kumar 1998b) and an optimization genetic algorithm previously developed (Iglesias-Rey et al. 2016).

Once the segmentation of the North and South I areas had been defined, it was possible to define the size that each of the DMAs would have in the rest of the network. Initially, the average demand inside a DMA was calculated from the target number of DMAs (15) and the average demand of the network (1924 l/s) obtaining 130 l/s in each DMA. Since the number of possible solutions up to this point was very small, the average size of the DMAs obtained after the engineering judgment approach was used to set the base demand for the remainder of the DMAs. This base demand allowed for defining the number of DMAs for each of the areas. In this case, the average size of the DMAs defined in the areas South II, North I and North II was 24 l/s. However, some of these DMAs were very small due to the need for avoiding pressures bigger than 60 m. If these DMAs are ignored, the average size of the standard DMAs rises to 36 l/s. After applying the optimization algorithm for different number of DMAs, the best partitioning for area South I was a division into 6 DMAs, with an average demand of 44 l/s. As a consequence, these values were used from this point on when defining the DMAs. The problem with them comes from the number of DMAs that will be defined. As the total demand is 1924 l/s, the number of DMAs after the partition will be above 50, much more than the intended 15. The final solution defined 59 DMAs.

In addition, METIS allows weighting factors for both nodes and edges (lines in the model). Demands at the nodes were used as their weight. With respect to the pipes, the network presents problems of low pressures at the moments of peak demand. This implies to ensure that the biggest pipes should remain opened since they have a great transport capacity. METIS define which pipes must be closed in order to isolate a DMA. For this reason, a large weight was assigned to these lines in order to avoid their closure. Depending on the location, pipes were weighted based on their diameter (prioritizing small or big pipes depending on the situation). Finally, once the parameters for the problem were defined, the METIS partitioning algorithm was used to define DMAs with equally distributed demands and the pipes suitable to become the boundaries between DMAs.

From this preliminary DMA selection, an optimization of the network operation was performed based on the following objective function:

$$F=\sum_{i=1}^{N_B} C_{B,i}(D_i)+PU_{net}+\lambda_0+\lambda_1\sum_{i=1}^{N_{DMA}}\left(N_{in}-2\right)+\lambda_2\sum_{i=1}^{N}\left(p_{\max,i}-p_{\max}\right)+\lambda_3\sum_{i=1}^{N}\left(p_{\min}-p_{\min,i}\right) \qquad (6)$$

In equation (6) the first sum represents the cost $C_{B,i}$ of potential pressure reducing valves (diameter D_i) to be installed at the entry to each of the sectors. Initially, N_B sector boundaries were available. Only those PRVs that finally remained closed represent a zero cost for this term.

The second term represents the uniformity of the pressure. All the other terms are penalty functions for non-verification of some of the constraints of the problem. Each of these restrictions has a multiplier to establish the order in which they should be defined. A staggered distribution of Lagrange multipliers was set in order to sort the solutions. From worst to best, this order would be the following: unfeasible solutions, feasible solutions with more than two inputs to the sector, and finally topologically correct solutions that violate the values of maximum or minimum pressure.

Thus, the first multiplier λ_0 represents the existence of a topological error in the definition of the network that leaves off a part of the network. This parameter takes an extremely high value ($\lambda_0=10^{10}$) in order to rule out unfeasible layouts in the solution. The second function represents the penalty for exceeding the maximum number of entries (a maximum of 2) in each of the N_{DMA} sectors. The value of this parameter is high but several orders of magnitude lower than the parameter λ_0 ($\lambda_1=10^6$). The last two terms of (6) represent the penalties that occur when one of the N nodes of the network has a minimum ($p_{min,i}$) or maximum ($p_{max,i}$) pressure outside the range defined by the values for minimum pressure ($p_{min}=15$) and maximum pressure ($p_{max}=60$) in the problem description. In this case, the parameters λ_2 and λ_3 are again several orders of magnitude lower ($\lambda_2=\lambda_3=10^3$).

After completing the PGA optimization process, we had a configuration for each area (Center and South I). These final solutions were analyzed in order to determine the need to change any of the initial parameters of the procedure. Thus, depending on the results, the following parameters could be modified:

- The definition of the main lines. The solution may generate a closure of one of the main pipes, which leads to a redefinition of the lines that must be out of sectors and those that are not.
- The definition of weights for the graph edges. These weights, as described below, depend on the incoming flow to each DMA. Therefore, after obtaining a solution, it might be necessary to redefine them. This leads to an iterative process, as shown in Figure 2.
- The definition of the number of DMAs. An analysis of the obtained solution may suggest the need to increase or decrease the number of sectors. Since the objective is to seek a uniformity of sizes for the different DMAs, redefining the size of the sectors may help in the search for the most appropriate solutions.

After completing the iteration process, a final solution of the network is provided once a model with all the areas calculated separately is established. A dynamic network simulation is performed, and then, a final adjustment can be made. This adjustment is based on engineering judgement and may be caused either by the violation of any of the rules of BWNDMA or by improving the rank of the solution in some evaluation criteria.

Weighting algorithm of the lines for graph definition.

In order to define the base graph for the DMAs definition, the METIS algorithm (Karypis and Kumar 1998a) was employed. This algorithm carries out a division of the original graph in a series of sectors. To make this division, it is necessary to define a weighting both for the vertices and the edges of the graph. For the vertices of the graph, the weighting parameter is the base demand of each node. Thus, it is intended that the sum of these base demands is uniformly distributed among the different DMAs. Conversely, the weighting of the edges is more complex. Apart from the reasons explained above, biggest pipes should not be the entrance to a DMA because of the price of the corresponding PRV. In contrast, undersized pipes may not have sufficient transport capacity to supply the sector.

The weighting methodology was based on the criteria of inflow to the DMA. Hence, the average intake flow to each sector defined a maximum diameter (D_{max}) and a minimum diameter (D_{min}) restricted by the maximum (j_{max}) and minimum (j_{min}) allowed values of the hydraulic gradient in the border pipes sector. These extreme values were based on engineering judgment, and they were taken between a maximum of j_{max} = 15 m/km and a minimum of j_{min} = 3 m/km. These values were calculated for velocities in the pipes ranging between 0.5 and 2.0 m/s. Depending on the number of entries to a DMA and its demand, the recommended diameter could be obtained. As a first approach, a demand value of 36 l/s for a DMA with only one entry led to diameters ranging between 102 mm for the maximum hydraulic gradient and 254 mm for the minimum. The partition algorithm should propose pipes with diameters between these two values. If this was not possible, other pipes could be used instead. As a rule of thumb small pipes are preferable to big ones as long as there are no pressure problems. Therefore, the weighting strategy should consider this. Transport capacity was used for this purpose using the cross sectional area, that is, the squared diameter, as a base value. Once the extreme values for the diameter of the boundary pipes were defined, smaller sized pipes are preferred since they involve a lower cost of installation for PRVs. Diameters lower than the minimum (that is, under 102 mm) had better weightings than any other pipe with diameter larger than the maximum. In the same sense, pipes with diameter larger than the maximum had higher weights. Thus, the weighting function can be expressed as:

$$\begin{array}{ll} \mathrm{D} > \mathrm{D}_{\mathrm{max}} & \frac{2 \cdot \mathrm{D}^2}{10000} \\ \mathrm{D}_{\mathrm{min}} < D < \mathrm{D}_{\mathrm{max}} & \frac{\mathrm{D}^2}{10000} \\ \mathrm{D} < \mathrm{D}_{\mathrm{min}} & \frac{\mathrm{D}_{\mathrm{max}}^2}{10000} + \frac{40000}{\mathrm{D}^2} \end{array} \tag{7}$$

In the equation (7) above, the weights have been chosen to account for all situations. As the METIS algorithm requires integer values for the weights in both vertices and edges, the results of equation (7) were converted into integers. As an example, Table 3 presents the values used for the assumption of an average demand size of 36 l/s.

Changes between the Rainy and Dry seasons

A priority aspect considered during the solution of the problem consists of reducing the number of maneuvers between the rainy and dry seasons. Given the configuration of the network shown in Figure 1, the only changes needed between seasons are the adjustments of the pumping flows. These flows are mainly supplied by water from the Cuza WTP. Thus, during the change from the rainy to dry seasons it is only necessary to make the following changes:

- Change the reference (FCV) that controls the filling of tank 12. This valve should allow a higher flow rate coming from Cuza WTP to compensate the loss of supply from the Fagua pumping station.
- Change the state (OPEN/CLOSED depending on the season) of two pipes in order to meet the Mohan flows Pumping Station with water from the Cuza WTP.

In short, the two modes of network operation (rainy and dry seasons) just change the network operation scheme.

RESULTS AND CONCLUSIONS

The application of the methodology described in Figure 2 and the application of small corrections based on engineering judgment finally allowed for obtaining the solution of the BWNDMA presented in Table 4. The definition of parameters for assessing pressure uniformity (PU), water age (WAnet) and demand similarity (DS) can be found in BWNDMA Committee (2016). It is a solution with 59 sectors, which are defined based on changing over 3 km of pipeline network (35 pipes in the model) and closing a large number of pipes. Likewise, the definition of the sectors requires the installation of 88 pressure reducing valves. However, since the only possible mode of operation for the network is through different tanks, it is necessary to increase the size of 5 tanks to ensure adequate hydraulic system behavior. These tanks have been highlighted in Table 2.

The BWNDMA provided an opportunity for researchers in hydraulic engineering to solve a real-world WDN optimization problem. The main conclusions of this work are presented as follows:

- The network presents a problem of water scarcity mainly during the dry season. This fact made it impossible to accomplish a solution with all the restrictions of the problem. Therefore, the maximum flow per water source was exceeded in order to satisfy all the demands of the network. As a result of the preliminary study, it was concluded that the Cuza WTP should exceed its maximum flow. Otherwise, the South area demand could not be supplied without violating pressure restrictions.
- There are two different situations corresponding with the two seasons. However, only the dry season presents problems for finding feasible solutions. In other words, if the dry season is solved, the whole system configuration will be solved.

- Despite the size of the network, the number of feasible solutions seems to be very small due to the complexity of the network in terms of maximum supplied flow in the dry season and great differences in elevation in some areas of the network. However, the procedure for defining DMAs based on METIS and PGA needed to use concepts based on engineering judgment. Thus, the weighting factors in both METIS (Table 3) and the objective function (equation 6) were based on engineering criteria.
- The network was divided into two different types of areas. One of them must be solved using engineering judgment to address some hydraulic complexities. The other one could be solved using a combination of the METIS partitioning algorithm and a pseudo-genetic optimization algorithm. To minimize the effect of the demand time pattern on water source flow, most of the demand is supplied from tanks. So, a detailed study of storage capacity was performed to determine the maximum demand each tank can supply. In the case of tanks 3, 5, 10, 12 and 14, their capacities were increased.
- The number of DMAs after the partition is 59, much larger than the intended 15. In exchange, both the demand and pressure similarity of the DMAs has been improved.
- The Mohan Pumping Station is not able to extract 206 l/s from its source. This situation makes the problem worse. In a real situation, changing the pumps could overcome part of the water scarcity problems.

To summarize, automated procedures for defining DMAs either based on graph partitions or on optimization algorithms have been shown to be effective. Nevertheless, in problems where the number of constraints is large and the solution space is small, the method requires the use of engineering judgment.

ACKNOWLEDGEMENTS

This work was supported by the Program Initiation into Research (Project 11140128) of the Comision Nacional de Investigación Científica y Tecnológica (Conicyt), Chile.

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Table 1. Data for the different operational areas considered.

Subarea	No. Nodes	Total Base Demand (l/s)	Sectorization method
North I	577	157.63	Engineering judgment
North II	1934	232.19	Engineering judgment
Center I	1238	258.46	Optimization
Center II	1752	505.39	Optimization
Center III	2499	359.66	Optimization
South I	552	78.28	Engineering judgment
South II	1166	311.24	Optimization
Municipality	1435	36.45	Engineering judgment

Table 2. Maximum flow that each tank can manage.

ID	Volume (m^3)	Tank expansion (m^3)					
		0	**500**	**1000**	**2000**	**3750**	**5000**
Tank 1	6480.0	310.6	334.5	358.5	406.4	490.3	550.2
Tank 3[a]	235.0	11.3	35.2	59.2	107.1	191.0	250.9
Tank 4	235.0	11.3	35.2	59.2	107.1	191.0	250.9
Tank 5[a]	1842.8	88.3	112.3	136.2	184.2	268.0	328.0
Tank 6	200.0	9.6	33.5	57.5	105.4	189.3	249.2
Tank 7	135.0	6.5	30.4	54.4	102.3	186.2	246.1
Tank 10[a]	700.0	33.5	57.5	81.5	129.4	213.3	273.2
Tank 11	8198.8	392.9	416.9	440.9	488.8	572.7	632.6
Tank 12[a]	9000.0	431.3	455.3	479.3	527.2	611.1	671.0
Tank 14[a]	3445.4	165.1	189.1	213.1	261.0	344.9	404.8
Tank 15	3628.2	173.9	197.9	221.8	269.7	353.6	413.5
Tank 16	10681.4	511.9	535.9	559.9	607.8	691.7	751.6
Tank 17	2706.5	129.7	153.7	177.6	225.6	309.4	369.3

[a] Tanks with size increased

Table 3. Edge weights for different diameters of the DMA border.

D (mm)	Weight
50	22
60	17
70	14
80	13
90	11
100	1
150	2

200	4
250	6
300	18
350	25
400	32
450	41
500	50
600	72
700	98

Table 4. Summary results.

Criteria	**Value**
Number of DMA	59
Pipe intervention costs (35 pipes)	$ 107,690.65
Tank intervention costs (5 tanks)	$ 350,206.00
PRV intervention costs (88 valves)	$ 161,013.00
Total costs	$ 618,909.65
Valve opening changes	0
Valve closure changes	2
FCV setting changes	1
Pressure Uniformity, PU (Rainy season)	235.93
Pressure Uniformity, PU (Dry season)	238.06
Water Age, WAnet (Rainy season)	1.46
Water Age, WAnet (Dry season)	1.44
Demand Similarity index (DS)	20.13

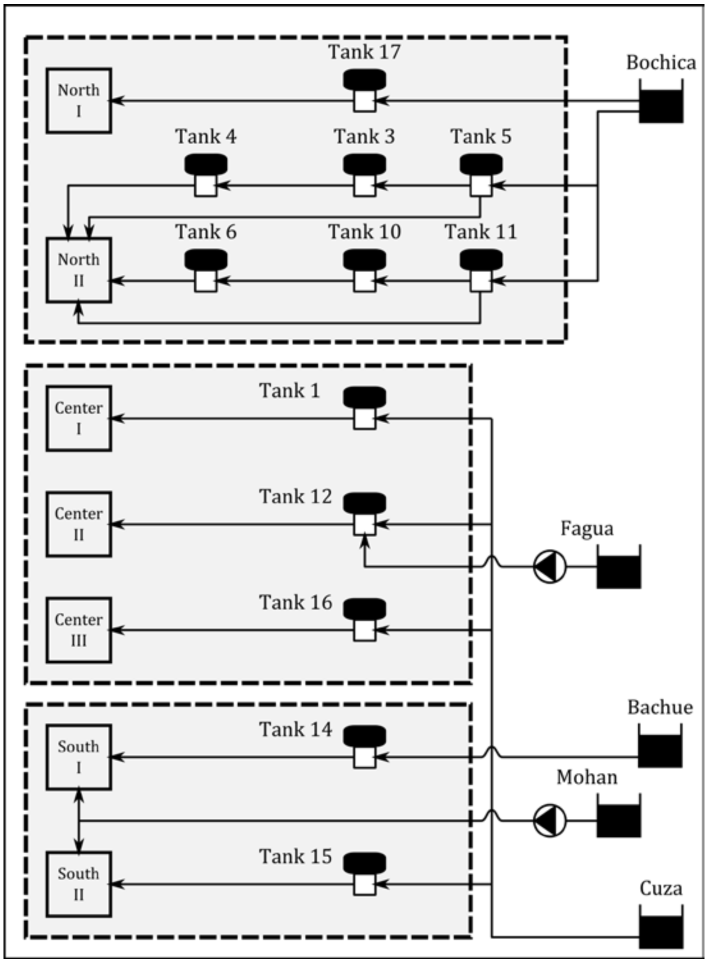

Figure 1. Division of the supply system in operational areas.

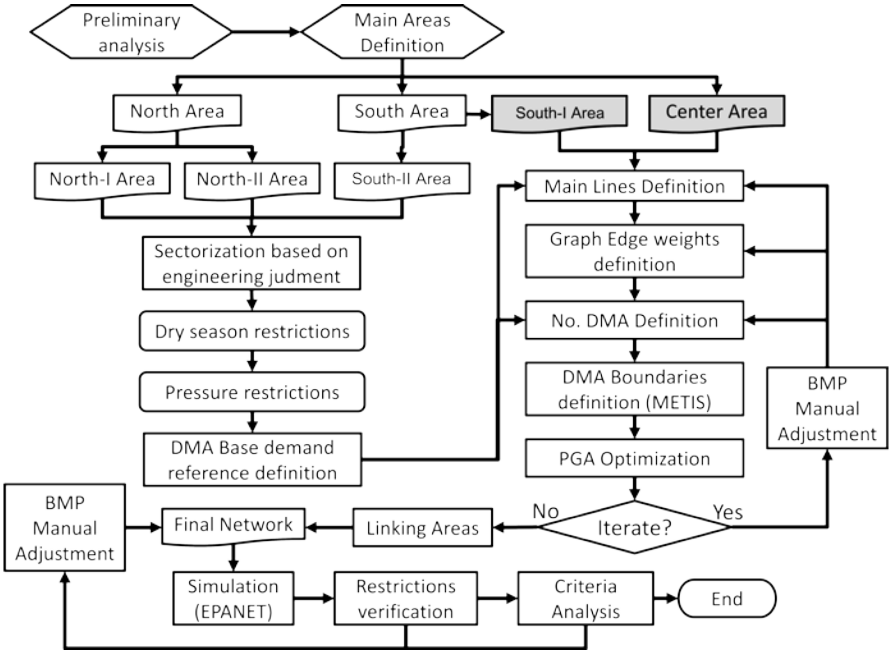

Figure 2. Methodology used to solve the BWDMA.

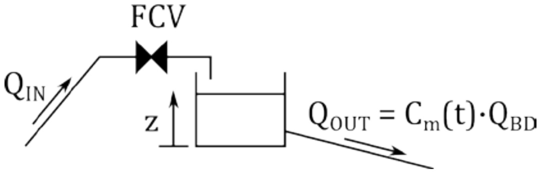

Figure 3. Operating diagram of a tank.

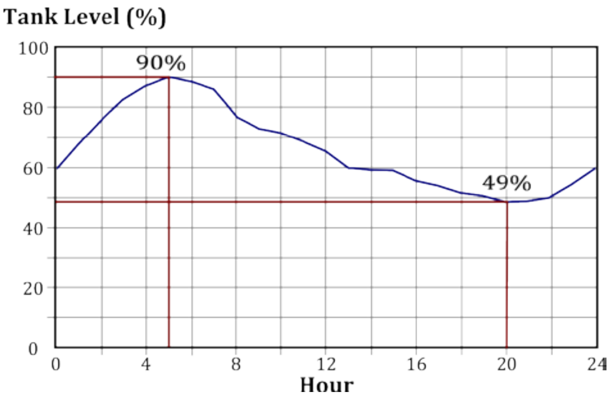

Figure 4. Evolution of tank level when average inflow and outflow are balanced.