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Abstract

Optimal operation of large water distribution systems (WDS) has always been a tedious task especially when combined with determination of district metered areas (DMAs). This paper presents a novel framework based on graph theory and optimisation models to design DMA configuration and identify optimal operation of large WDS for both dry and rainy seasons. The methodology comprise three main phases of preliminary analysis, DMA configuration and optimal operation. The preliminary analysis assists in identifying key features and potential bottlenecks in the WDS and hence narrow down the large number of decision variables. The second phase employs a graph theory approach to specify DMAs and adjust their configuration based on similarity of total water demands and pressure uniformity in DMAs. Third phase uses several consecutive single-objective and multi-objective optimisation models. The decision variables are pipe rehabilitation, tank upgrade, location of valves and pipes closure, and valve settings for each DMA. The objective functions are to minimise total annual cost of rehabilitation, water age and pressure uniformity. The proposed methodology is demonstrated through its application to large real-world WDS of E-Town. The results show that the proposed methodology can determine a desirable DMA configuration mainly supplied directly by trunk mains.

Keywords: District metered area; graph theory; optimal rehabilitation; water distribution systems.

Introduction

Population and economic growth around the globe is driving up water demand (UN 2005), thus increasing extra pressure on urban water infrastructures, especially municipal water distribution systems (WDS). These require redesign, rehabilitation plan and management of water infrastructure while considering technical, economic and security aspects of WDS. However, optimal redesign or rehabilitation plan of WDS, to maintain high level of service, is a time-consuming and tedious task especially in large WDS due to a heavy computational burden. In recent decades, many researchers have focused on various assessment criteria to improve the performance of WDS while using different optimisation approaches. Some examples are to maximise resilience (Farmani et al. 2005, Nafi and Kleiner 2009, Matthews et al. 2014), reliability (Farmani et al. 2006, Kapelan et al. 2005), minimise leakage (Araujo et al. 2006, Ali 2015, Shafiee et al. 2016, Vassiljev and Puust 2016, Giustolisi et al. 2016), greenhouse gas emissions (Rahmani et al. 2014), minimise both cost and energy (Rahmani et al. 2015; Ostfeld et al. 2013) and minimise both leakage and cost (Mahdavi et al. 2010).

Division of WDS into smaller zones, so-called District Metered Areas (DMAs), is a common approach particularly in the UK for a better management of water demands in WDS (Walski et al. 2003). Other applications of the DMA are to reduce water loss and pressure management (Thornton et al. 2008, Laucelli et al. 2017), protect network water quality (Di Nardo et al. 2013, Zhang et al. 2017) and improve optimal rehabilitation of WDS (Muhammed et al. 2017). Each DMA is isolated by valves and connected to the rest of WDS by a small number of inlet and outlet water mains. Ferrari et al. (2014) suggested the following criteria in DMA configuration: DMAs should be independent from each other (i.e. preferably fed directly by water sources) with similar size, elevation and total demands as much as possible. DMA design entails specifying DMA boundaries, settings of isolation valves and water mains feeding a DMA. Various techniques have been developed for DMA design such as graph theory (Ferrari et al. 2014, Scarpa et al. 2016, Zhang et al. 2017) and clustering technique (Diao et al. 2014). These techniques are based on some heuristic procedures to define isolated DMAs (i.e. fully independent districts) or reduce the influence of DMAs on each other (e.g. for decreasing contaminant propagation) in WDS (Di Nardo et al. 2013). Gomes at al. (2012) proposed a methodology based on graph concepts to automatically generate a reasonable number of DMAs and identify optimal locations of DMA inlet and outlet points and boundary valves. Scarpa et al. (2016) proposed another method in which the network is first divided into primary areas (e.g. based on supply sources) and then the areas are adjusted based on possible DMA design level scenarios. Alvisi and Franchini (2014) suggested a heuristic methodology for DMAs configuration using graph theory and hydraulic simulation. Giustolisi and Ridolfi (2014) developed segmentation of WDS based on a modularity-based approach by using a multi-objective optimisation framework strategy. Diao et al. (2015) proposed another approach based on decomposition of the whole WDS into twin-hierarchy pipeline structure which includes backbone mains and communities. Laucelli et al. (2017) introduced a two-phase design strategy for optimal DMA design to minimise the number of flow meters and background leakage. Giustolisi et al. (2017) presented a new approach for network structure classification of WDS based on 'neighbourhood' degree which can be used for DMA configuration.

The most common criteria for DMA design applied in the above studies are DMA size, DMA boundary, topological features and their connectivity to transmission main. These criteria should be considered concurrently with satisfying hydraulic conditions and optimising various objectives of the system (e.g. total cost and water quality) in WDS. This can result in a challenging task especially in large WDS. Despite a plethora of previously developed methodologies, to the best of the authors' knowledge, only few studies of DMA design handled large WDS with multiple criteria (Laucelli et al. 2017). This paper presents a new framework for both DMA design and optimal operation of large WDS based on graph theory and optimisation techniques. The methodology aims to concurrently generate a desirable and new DMA configuration while attaining hydraulic requirements, minimising rehabilitation costs in WDS. The main objectives considered in the optimisation include the desirable number of DMAs, total demand similarity and pressure uniformity between DMAs, desirable water quality in WDS and minimum number of operational changes between dry and rainy seasons. A brief overview of the problem description and details of the proposed methodology are described in the next sections followed by the results and discussion section and finally concluding remarks.

Problem Description

The case study used in this paper is called 'E-Town' which is a large and real water distribution system in Colombia. It includes almost 11,000 nodes and 14,000 pipes with an approximate length of 900 kilometres. The E-Town WDS was first presented in the Battle of Water Networks District Metered Areas (BWNDMA) problem at the Water Distribution System Analysis conference (BWNDMA 2016). The related water utility aims to develop a new DMA configuration which improves the performance by facilitating efficient use of the current infrastructure with respect to the limited water availability throughout the year. The hydraulic model is simulated for a two-week period, one week representing the rainy season and the other for the dry season. The specific objectives of the water utility are divided into three groups: first is related to the DMA design features (i.e. the number of DMAs, total water demand similarity in DMAs); second is the hydraulic performance and operation (i.e. water age of WDS, pressure uniformity in DMAs and the number of the operational changes between the two seasons) and third is the total annual cost of rehabilitation. Pressure uniformity and water age are calculated for both seasons separately. A summary of the constraints proposed by the water utility is as follows: (1) maximum and minimum pressure heads equal to 60m and 15m, respectively for demand nodes inside DMAs, and positive pressure heads for non-demand nodes; (2) the water level in tanks at the end of simulation must be at least the same water level at the beginning of the simulation period; (3) tank volume can change between 10 and 90 percent of the tank capacity; (4) each DMA is allowed to have a maximum of two inlets controlled by Pressure Reducing Valves (PRVs); (5) pressure setting of all valves used in the WDS can change only between the seasons; (6) addition of new tanks is allowed only adjacent to existing tanks; (7) addition of hydraulic controls to the existing pump stations is allowed but variable speed pumps or time controlled pumps are not allowed; (8) the number of DMAs must be at least 15 and the ideal number is also 15. The replacement of pipes is restricted to pipes with diameters larger than 152mm. Further details of problem description can be found on the BWNDMA website (Saldarriaga 2016) .

Methodology

A framework is proposed for optimal operation of WDS with new DMA configuration, which consists of three sequential main phases: preliminary analysis, DMA configuration and optimal operation. These phases are made up of 11 steps as shown in Fig. 1. The first phase generally assists the designer in identifying key characteristics of the system and potential bottlenecks and critical parts. Configuration of DMAs is conducted through four sequential steps in the second phase. In the third phase, first a feasible hydraulic model is identified and then optimal operation of WDS is generated. The software tools used in this paper are Gephi (2014) for creation of DMAs and EPANET (Rossman 2000) for hydraulic and water quality model simulation. The single objective and NSGA-II multi-objective evolutionary algorithms (Deb et al, 2002) are also used for optimisation of WDS operation. The details of each step are provided in the following.

Step 1: This step comprises the preliminary analysis of WDS in which key features and bottlenecks of WDS are recognised. These can include investigation of supply and demand in the WDS in both seasons; identification of pipes with high pressure head loss (e.g. old pipes with a large roughness value or pipes with a small diameter); topological analysis across the system. This is particularly important for the current configuration of the E-Town WDS which fails to supply future water demands. More specifically, the analysis of hourly water supply and demand of the system can reveal additional tanks required to accommodate the fluctuations in diurnal water demands. The preliminary analysis can also identify any odd behaviour of the elements (e.g. filling and emptying tanks) that may lead to unusual or large simulation run time in the hydraulic model. Also, zones or nodes in the system with uncommon characteristics (e.g. adjacent nodes with relatively large difference in elevation) can be spotted which need further attention within hydraulic simulation (e.g. maintaining concurrently minimum and maximum pressure heads for those nodes). All this information can be useful when designing DMA or determining decision variables for rehabilitation options.

Step 2: This step entails clustering the WDS to identify DMAs using a graph theory technique based on the classical modularity-based method proposed by Clauset et al. (2004). This method is used due to its competency for high speed and reliability in decomposition of large-scale complex systems (Newman, 2006). The method is implemented by using an open source and free software called "Gephi" which is widely used for graph network visualisation and manipulation

(Gephi, 2014). The WDS clustering is carried out in two steps (Muhammed et al. 2017): 1) the WDS is mapped into an undirected graph in which the vertices represent nodes, reservoirs and tanks and the edges represent pipes, pumps and valves; 2) the graph is divided into clusters of vertices characterised by stronger interconnections based on the modularity index, a metric to be maximised during clustering, which is defined as:

$$Q = \frac{1}{2m}\sum_{\upsilon\omega}\left[A_{\upsilon\omega} - \frac{k_\upsilon k_\omega}{2m}\right]\delta(c_\upsilon, c_\omega) \tag{1}$$

where $A_{\upsilon\omega}$ = an element of the adjacency matrix of the network; $k_\upsilon = \sum_\omega A_{\upsilon\omega}$ = the sum of the number of edges connected to vertex υ; c_υ = the cluster to which vertex υ belongs, $\delta(c_\upsilon, c_\omega)$=1 if $c_\upsilon = c_\omega$, otherwise it is 0 and $m = \frac{1}{2}\sum_{\upsilon\omega} A_{\upsilon\omega}$ = the number of edges in the graph. The required number of clusters is provided in Gephi by the "Resolution" parameter in the modularity settings. The default value of "Resolution" is set to one, higher values lead to fewer clusters and vice versa. As the level of decomposition is case-specific for any WDS analysis, an appropriate level of decomposition, e.g. number of clusters, could be discovered based on trial and error approach using different resolution values. Also, note that PRVs and gate valves which can be used for entrance and exit points of DMAs are defined close to the end of pipes (i.e. nodal points) in the EPANET model as specified by the water utility. The technique suggested by Giustolisi and Ridolfi (2014) is also an alternative approach which can be applied for dividing the WDS into segments based on the attributes of nodes and pipes.

Step 3: This step identifies transmission/trunk mains by which DMAs are isolated from each other. In other words, the DMAs are connected to the system through the trunk mains. In this study, trunk mains are defined based on three key features: direct link between a water source and a tank, flow rate capacity and pipe size. Thus, all ordinary pipes in the boundary of a DMA connected to other DMAs are closed. This step also aims to maximise the number of DMAs fed directly by water sources through trunk mains and hence minimise the number of DMAs fed from other DMAs (Ferrari et al. 2014).

Step 4: This step amends the boundary of DMAs further on a trial and error basis to reduce the discrepancy of the total water demand of DMAs as much as possible. This entails identifying the nodes adjacent to the boundaries of DMAs with larger total water demand, which can result in a more similarity of total water demand between neighbouring DMAs when moved from one DMA to another. These modifications are made by either further decomposition of larger DMAs or combination of two smaller DMAs while keeping the total number of DMAs equal to 15. Attributes of nodes including elevation and demand are also considered in the amendment of DMA boundaries. The total water demand of each DMA is automatically calculated using a MATLAB code, after each modification to boundaries of DMAs, in order to evaluate the DMA similarity indicator. This procedure is repeated until reasonable convergence is obtained in the similarity of the total water demand of DMAs.

Step 5: Based on the constraint of maximum two PRVs at each DMA inlet (BWNDMA 2016), potential locations for installing PRVs at DMA boundaries are specified in this step. As each DMA can be fed by either a trunk main or a neighbourhood DMA, the potential PRV locations are either on the pipes connecting DMAs to trunk mains or pipes between neighbouring DMAs. The potential PRV locations are considered as decision variables in the optimisation problems in the next phase. The pipes which are not selected by the optimisation problems as optimum location for PRVs at DMA boundaries will be closed.

Step 6: This step entails developing and running a genetic algorithm-based single-objective optimisation problem to identify a feasible solution for the hydraulic simulation model (i.e. model with no negative pressure). The objective is to minimise negative pressures (i.e. effectively with the aim of eliminating negative pressures) in the hydraulic simulation model as:

$$Min(F_1) = \sum_{j=1}^{M}\sum_{i=1}^{N}\left|P_{ij}\right| \quad if\ P_{ij} < 0 \qquad (2)$$

where P_{ij} = the deficit pressure at junction i at time j; M and N = number of simulation time steps and total number of system junctions respectively. Due to the limited water availability during the dry season, it is more challenging to identify a feasible solution in that season. Therefore, the optimisation problem in this step considers only the rainy season. The decision variables (Table 1) include the options for tank upgrade, PRV locations and settings, flow control valve (FCV) settings, pipe closure and pipe rehabilitation. In order to reduce optimisation search space, a small percentage of pipes which may have more impact on achieving a feasible solution are selected as decision variables. The criteria used in the selection are pipes with large contribution to pressure head loss which is a function of pipe friction coefficient, pipe diameter and pipe length. This information is provided in the first step where preliminary analysis is carried out. The constraints in both optimisation problems are minimum and maximum pressure heads and tank water levels specified by the water utility (BWNDMA 2016).

Step 7: If the optimisation problem in step 6 fails to find a feasible solution (i.e. no solution without negative pressures) after a specific number of generations, this step is employed to modify decision variables and repeat step 6 to speed up the convergence towards a feasible solution. The modification of decision variables is based on the assessment of the last solution obtained in step

6. More specifically, new decision variables are identified in the areas with pressure deficit or negative pressures. The new decision variables include critical pipes such as trunk mains, pipes located in the upstream of the areas with negative pressures, pipes in the paths from water sources to tanks that supply those areas, and from tanks to DMAs with pressure deficit. The new decision variables are added to the list of existing decision variables in step 6. The optimisation model in step 6 is run again with the new set of decision variables until a feasible solution is obtained for the rainy season.

Steps 8 and 9: The single-objective optimisation problem defined in step 6 is used in step 8 in which both rainy and dry seasons are considered to obtain a feasible solution for both seasons simultaneously. The dry season period is added to the optimisation problem at this stage. Decision variables of the optimisation problem are defined similar to step 6 (Table 1). The decision variables are updated in step 9 if no feasible solution generated after a number of generations and the optimisation run is repeated in a loop until a feasible solution is finally obtained.

Step 10: This step entails a multi-objective optimisation problem with the aim of improving the performance of the WDS and providing a set of optimal solutions as a trade-off between the objectives specified by the water utility. This step considers a full set of decision variables specified in both seasons. The objectives are to minimise 1) sum of the pressure uniformity indicator in both seasons (f_1) (Eq. 3); 2) sum of the water age indicator in both seasons (f_2) (Eq. 5); 3) total annual costs of rehabilitation (f_3) (Eq. 7).

$$Min\ f_1 = PU_{net} = \sum_{k=1}^{M_S}\sum_{j=1}^{M_{time}}\left[\frac{1}{N_{junc}}\sum_{i=1}^{N_{junc}}\frac{(P_{ijk}-P_{\min})}{P_{\min}} + \frac{\sqrt{\sum_{i=1}^{N_{junc}}\frac{(P_{ijk}-P_{avjk})^2}{N}}}{P_{avjk}}\right] \qquad (3)$$

$$P_{avj} = \frac{\sum_{i=1}^{N_{junc}} P_{ij}}{N_{junc}} \tag{4}$$

$$Min\ f_2 = WA_{net} = \sum_{k=1}^{M_s} \frac{\sum_{i=1}^{N_{junc}} \sum_{j=1}^{M_{time}} k_{ijk} Q_{dem,ijk} (WA_{ijk} - WA_{th})}{\sum_{i=1}^{N_{junc}} \sum_{j=1}^{M_{time}} Q_{dem,ijk}} \tag{5}$$

$$k_{ijk} = \begin{cases} 1, & WA_{ijk} \geq WA_{th} \\ 0, & WA_{ijk} > WA_{th} \end{cases} \tag{6}$$

$$Min\ f_3 = CC_{net} = \sum_{i=1}^{NN} C_i L_i + \sum_{j=1}^{MM} K_j + \sum_{k=1}^{OO} V_k \tag{7}$$

where PU_{net}= the system pressure uniformity; P_{ijk} = the pressure at junction i, time j and season k; P_{min} =the minimum allowable pressure (15 m); P_{avjk} = the average pressure in the system at time j and season k; WA_{ijk} = the water age at junction i, time j and season k in hour; WA_{net} = the system water age in hour; WA_{th}= the water age threshold (60 hours); $Q_{dem,ijk}$ = the demand at junction i, time j and season k; CC_{net} = the total system capital cost; C_i , L_i = the unit cost and total length of rehabilitated pipe i; K_j = the cost of installed valve j; V_k = the cost of added volume in tank k; NN, MM and OO= total number of rehabilitated pipes, installed valves and tanks with added volume in both seasons; N_{junc} =the number of system junctions; M_{time} = the number of simulation time steps (equal to 168 one-hour steps as the extended period simulation time of one week) and M_s = the number of seasons (i.e. 2 including rainy and dry). Note that pressure uniformity and water age indicators defined above are evaluated for the junctions with non-zero water demand. Also, the cost function (f_3) includes annual capital investment for rehabilitation only (i.e. not operational costs such as energy costs) as set by the water utility (BWNDMA 2016).

Results and discussion

The proposed methodology is demonstrated here for the E-Town WDS. Water supply and demand in the WDS is analysed in the first step. The analysis shows that the capacity of water supply is 2,850 l/s for rainy (including two water sources) and 2,080 l/s for dry season (including the same two water sources plus two pump stations). However, the average water demand is 1,917 l/s with a peak value of 2,957 l/s. The reduced capacity of water supply in the dry season requires some operational changes when moving to the next season. By comparing the values of peak demand and supply, it is evident that the WDS needs to effectively use the capacity of storage tanks to store water for peak demands.

Graph theory method is used to generate 15 DMAs in the WDS in step 2 to comply with the desirable number of DMAs specified by the water utility (BWNDMA 2016). A total of 968 pipes are identified in step 3 as transmission/trunk mains, with a pipe diameter ranging between 300 and 1524 mm, which are around 9% of the total WDS pipes. Modification of the DMA configuration is made for 10 DMAs in step 4 based on nodal elevations and layout of trunk mains to improve the similarity of total water demand of DMAs. More specifically, some DMAs with large total water demand are split into several new DMAs while some DMAs with small total water demand are merged together (see some examples in Fig. 2). The final configuration of DMAs is obtained in step 4 (Fig. 3) with the main characteristics given in Table 2. As can be seen, three distinct groups of similar total water demand can be observed including 6 DMAs (i.e. 1, 2, 3, 4, 5 and 6) with a weekly average demand of around 85,000 m^3, 2 DMAs (i.e. 7 and 9) with slightly larger demand (around 97,000 m^3) and 7 DMAs (i.e. 8, 10, 11, 12, 13, 14 and 15) with relatively smaller demand (around 70,000 m^3). Further investigation of the large total water demand in DMAs 7 and 9 revealed that there are a few small branches that cannot be separated and combined with other DMAs due to their specific geographical locations. A total of 135 potential locations are also identified in step 5 as potential locations for PRV installation at the entrance/boundary points of DMAs.

The parameters undertaken for the single and multi-objective optimisation in steps 6-10 are as follows: a population size of 30 for steps 6-8 and 50 for step 10, two-point crossover with a probability rate of 0.95, and a mutation by gene with a probability rate equal to the inverse of the chromosome length. The optimisation runs were carried out on a computer with following specifications: Intel(R) Core(TM) i5-2410M CPU @2.30GHz with 4 GB of installed memory (RAM). Number of generations conducted in steps 6, 8 and 10 were 500, 600 and 500, respectively. Each simulation run took almost 0.5, 1 and 6 seconds in steps 6, 8 and 10 respectively. Furthermore, adding new decision variables in steps 7 and 9 took almost around 30 hours for each step.

Fig. 4 shows the Pareto fronts of optimal solutions obtained from the multi-objective optimisation runs described in step 10. The trade-offs between the three objective functions indicate a strong correlation between the annual capital cost and the pressure uniformity (PU) indicator. In other words, this shows that the PU can be improved in the system through the capital investment in pipeline rehabilitation, increasing number of DMAs (Salomons, et al. 2017) and upgrading tanks. More specifically, pipeline rehabilitation can result in decreasing pressure head losses and hence pressure differences in DMAs. Upgrading tanks can also prevent from pressure differences during the simulation especially during peak times. Any solution in these Pareto fronts can be chosen and introduced as the selected optimal solution. The selected solution is better than 80 percent of solutions on Pareto front with respect to the cost and is the closest to the best values of the other two objective functions (This solution is shown by a black square in the Pareto fronts of the figure).

A feasible solution is generated following optimisation model runs in steps 6 and 8. In step 10, a solution with maximum two PRVs at entry points of each DMA is selected from Pareto fronts generated by the multiobjective optimisation model. Table 2 provides characteristics of the optimal solution obtained in these steps for both seasons. The connections between the DMAs and trunk mains during the rainy season shows that 7 DMAs are fully independent (i.e. supplied directly from the water sources through trunk mains only), 4 DMAs are partially independent (i.e. supplied form both trunk mains and other DMAs) and 4 DMAs are fed by adjacent DMAs. During the dry season, the only switch of water supply happens in DMA1 where its water sources change from partially independent to fully independent (i.e. both connections are fed through trunk mains). This shows that the majority of the DMAs in the solution take the advantage of direct supply from the trunk mains as desired by water utilities (Ferrari et al. 2014).

Table 3 also shows the characteristics of the 17 tanks in the selected solution. Out of 17 tanks, only 8 tanks need to be upgraded with added volume ranging between 1000 and 5750 m^3. The optimal solution shows that the tanks have a key role in providing the balance between diurnal water demand and supply in the WDS. Out of 17 existing tanks, 7 tanks remain inactive always and for 3 more (Tanks 1, 11 and 17) become active only during the dry season due to limited water available in water sources during peak times. Also, upgrading four tanks with a large capacity (Tanks 1, 12, 14 and 16) seems to be quite important as each of them supplies water for more than one DMA directly or indirectly. This can be seen by comparing the associated DMAs of these tanks (DMAs 1, 5, 8, 9, 10, 11 and 12) in Table 3 and their connection with other DMAs (DMAs 2, 4, 6 and 13) in Table 2. Furthermore, operation of some tanks (e.g. Tanks 5 and 11) have had a key role in eliminating negative pressures in the relevant DMAs during peak times in the dry season. More specifically, Tanks 5 and 11 are at the highest elevation compared to other points in the related DMAs (i.e. 14 and 15) and elevation of almost all other tanks in the system. In addition, most of the topography of the DMAs is flat, except DMAs 1, 14 and 15 which have relatively high elevation differences. In particular, DMA 14 has the highest point in the system and hence providing a balance of water supply between lowest and highest elevation points is difficult. Therefore, tanks 5 and 11 can guarantee that the highest demand points are supplied with sufficient pressure during peak times.

Due to the changes in both flow capacity and type of water sources between the two seasons, FCV settings located at the entrance of tanks have vital role in providing the balance between water demand and supply during peak times. For instance, Tank 14 is fed by only one water source (WTP1) with a maximum flow of 800 l/s during the rainy season but the water sources feeding the tank changes in the dry season to two sources (WTP1 and a pump station of SP1) with a total of maximum flow of 626 l/s. The results show that FCV settings in Tank 14 is reduced from 233.2 l/s in the rainy season to 161 l/s in the dry season. This can be due to the fact that the network (i.e. related DMAs) can be directly fed by Mohan pump station (SP1) which can also provide sufficient pressure head. However, the only water source available in the rainy season (i.e. WTP1) is unable to provide sufficient pressure head for directly feeding the network during peak times and hence Tank 15 needs to be used more by filling more water through a larger FCV setting. Similarly, the settings of other FCVs should be more limited due mainly to the lack of available water and benefiting from the two pump stations (SP1 and SP2) working only during the dry season.

Table 4 provides objective function values of the selected solution for the two seasons. Interestingly, the water age indicator in the dry season is lower than that in the rainy season. This can be attributed to the fact that the two additional water sources (i.e. pump stations) in the dry season can directly feed part of water demands which can reduce water age. However, the tanks are more used in the rainy season for saving during off-peak times and feeding the network during peak times. In addition, the two additional water sources used in the dry season are closer to the demand points that can also affect the water age indicator. Similarly, the pressure uniformity indicator in the dry season is lower than that in the rainy season. This can also be due to using pumped storage system in the rainy season which can lead to more fluctuations of pressure in the system compared to more pumped system in the dry season. A large proportion of the total cost in the selected solution is allocated to pipe rehabilitation (i.e. 50%) compared to approximately 38% for upgrading tanks and 12% for installing new PRVs. More specifically, the proportion of pipe rehabilitation components in the total rehabilitation cost is 61% for replacement of 139 pipes with a total length of 14.3km and 39% for duplication of 81 pipes with a total length of 6.3km. In addition, the cost of PRV installation is associated with installation of 28 new PRVs. The total number of operational changes from rainy to dry season is 29 (i.e. 10 FCV settings, 4 tank status and 15 PRV settings as shown in Tables 2 and 3). This is mainly due to adaption of the system to the changes in the capacity and type of water sources.

Conclusions

A new framework was presented, comprising three phases based on preliminary analysis, a graph theory and single-and multi-objective optimisation approaches, to identify a desirable DMA configuration and optimal operation for a real-world WDS. The preliminary analysis provided an invaluable engineering judgement to identify key features and potential bottlenecks in WDS for clustering and optimisation techniques. The optimal operation was obtained based on the requirements of the water utility (i.e. minimum total cost of rehabilitation, pressure uniformity and water age indicators in rainy and dry seasons and desirable number of DMAs). The design variables were optimal site locations for PRV installation and closing pipes in the boundary of DMAs, pipe rehabilitation, tank upgrade and FCV settings at the entrance of tanks. The performance of the proposed methodology was demonstrated by application to the E-Town WDS. The selected solution obtained from the multi-objective optimisation problem includes desirable number of 15 DMAs which can be classified under three groups of similar total water demand and majority of them are fed by trunk mains. The following can be noted from the obtained solution in the case study:

- Combination of engineering judgement, graph theory and optimisation techniques is an efficient method to handle DMA design and optimal operation of large real-world WDS simultaneously.
- A desirable configuration of DMAs with similar demands mainly supplied by trunk mains was obtained based on the criteria specified by the water utility.
- Due to increasing water demands, the system needed to efficiently use the tanks by filling tanks during off-peak times and feeding the system during peak times when the total demand was much larger than the total supply from water sources. Therefore upgrading tanks which feed specific DMAs was a key factor to provide supply demand balance in both seasons especially during peak times.

- Due to the major changes in the capacity and the type of water supply sources between the two seasons, the optimal operation of the WDS uses more capacity of the storage tanks in the rainy season whilst the storage tanks is used less in the dry season and it is more replaced with direct pump from additional pump stations.
- As a result of the above changes, settings of FCVs in most of the tanks is reduced to lower values in the dry season as water demands partially supplied directly by pump stations. This results in lower water age in the dry season than that in the rainy season when the water is more stored in the tanks.

The classical modularity index in this paper was implemented by using Gephi tool for clustering of WDS. Even with large number of nodes in the WDS, the tool was able to cluster the WDS fast and reliable. The modified modularity index proposed by Giustolisi and Ridolfi (2014) is also recommended for future works as an appropriate alternative clustering technique in WDS especially when a multi-criteria framework strategy is required. In addition, management of background leakage should be an integral part of assessment criteria when dealing with real world WDS. Modelling of background leakage can be simply carried out by a surrogate orifice at the nodes in EPANET software but a big picture of total leakage is usually overlooked which can be adjusted and verified with the total percentage of leakage by using a simple mass balance approach.

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