

Optimal Operation of Water Distribution Systems using a Graph Theory-Based Configuration of District Metered Areas

3

4 Farshid Rahmani, Research Graduate, Department of Civil and Environmental Engineering,
5 Amirkabir University of Technology, Tehran, Iran, email: farshid.rahmani@aut.ac.ir

6 Karwan Muhammed, Lecturer, College of engineering, Irrigation Department, University of
7 Sulaimani, Sulaymania, Kurdistan Region, Iraq, E-mail: kam223@exeter.ac.uk (Lecturer, College of
8 engineering, Department of Civil Engineering, Komar University of Science and Technology,
9 Sulaymania, Kurdistan Region, Iraq, E-mail: karwan.muhammed@komar.edu.iq

10 Kourosh Behzadian*, PhD, CEng, Senior Lecturer, School of Computing and Engineering,
11 University of West London, London, W5 5RF, UK, E-mail: kourosh.behzadian@uwl.ac.uk

12 Raziye Farmani, PhD, Associate Professor, Centre for Water systems, Department of
13 Engineering, University of Exeter, Exeter, EX4 4QF, UK, E-mail: r.farmani@exeter.ac.uk

14

15

16

17

18

19 * Corresponding author, Email: kourosh.behzadian@uwl.ac.uk, Tel: +44 (0) 20 8231 2466

Optimal Operation of Water Distribution Systems using a Graph Theory-Based Configuration of District Metered Areas

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.

40 **Introduction**

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

54 Division of WDS into smaller zones, so-called District Metered Areas (DMAs), is a common
55 approach particularly in the UK for a better management of water demands in WDS (Walski et al.
56 2003). Other applications of the DMA are to reduce water loss and pressure management
57 (Thornton et al. 2008, Laucelli et al. 2017), protect network water quality (Di Nardo et al. 2013,
58 Zhang et al. 2017) and improve optimal rehabilitation of WDS (Muhammed et al. 2017). Each
59 DMA is isolated by valves and connected to the rest of WDS by a small number of inlet and outlet
60 water mains. Ferrari et al. (2014) suggested the following criteria in DMA configuration: DMAs
61 should be independent from each other (i.e. preferably fed directly by water sources) with similar
62 size, elevation and total demands as much as possible. DMA design entails specifying DMA

63 boundaries, settings of isolation valves and water mains feeding a DMA. Various techniques have
64 been developed for DMA design such as graph theory (Ferrari et al. 2014, Scarpa et al. 2016,
65 Zhang et al. 2017) and clustering technique (Diao et al. 2014). These techniques are based on some
66 heuristic procedures to define isolated DMAs (i.e. fully independent districts) or reduce the
67 influence of DMAs on each other (e.g. for decreasing contaminant propagation) in WDS (Di Nardo
68 et al. 2013). Gomes et al. (2012) proposed a methodology based on graph concepts to automatically
69 generate a reasonable number of DMAs and identify optimal locations of DMA inlet and outlet
70 points and boundary valves. Scarpa et al. (2016) proposed another method in which the network
71 is first divided into primary areas (e.g. based on supply sources) and then the areas are adjusted
72 based on possible DMA design level scenarios. Alvisi and Franchini (2014) suggested a heuristic
73 methodology for DMAs configuration using graph theory and hydraulic simulation. Giustolisi and
74 Ridolfi (2014) developed segmentation of WDS based on a modularity-based approach by using a
75 multi-objective optimisation framework strategy. Diao et al. (2015) proposed another approach
76 based on decomposition of the whole WDS into twin-hierarchy pipeline structure which includes
77 backbone mains and communities. Laucelli et al. (2017) introduced a two-phase design strategy
78 for optimal DMA design to minimise the number of flow meters and background leakage.
79 Giustolisi et al. (2017) presented a new approach for network structure classification of WDS
80 based on 'neighbourhood' degree which can be used for DMA configuration.

81 The most common criteria for DMA design applied in the above studies are DMA size, DMA
82 boundary, topological features and their connectivity to transmission main. These criteria should
83 be considered concurrently with satisfying hydraulic conditions and optimising various objectives
84 of the system (e.g. total cost and water quality) in WDS. This can result in a challenging task
85 especially in large WDS. Despite a plethora of previously developed methodologies, to the best of

86 the authors' knowledge, only few studies of DMA design handled large WDS with multiple criteria
87 (Laucelli et al. 2017). This paper presents a new framework for both DMA design and optimal
88 operation of large WDS based on graph theory and optimisation techniques. The methodology
89 aims to concurrently generate a desirable and new DMA configuration while attaining hydraulic
90 requirements, minimising rehabilitation costs in WDS. The main objectives considered in the
91 optimisation include the desirable number of DMAs, total demand similarity and pressure
92 uniformity between DMAs, desirable water quality in WDS and minimum number of operational
93 changes between dry and rainy seasons. A brief overview of the problem description and details
94 of the proposed methodology are described in the next sections followed by the results and
95 discussion section and finally concluding remarks.

96

97 **Problem Description**

98 The case study used in this paper is called 'E-Town' which is a large and real water distribution
99 system in Colombia. It includes almost 11,000 nodes and 14,000 pipes with an approximate length
100 of 900 kilometres. The E-Town WDS was first presented in the Battle of Water Networks District
101 Metered Areas (BWNDMA) problem at the Water Distribution System Analysis conference
102 (BWNDMA 2016). The related water utility aims to develop a new DMA configuration which
103 improves the performance by facilitating efficient use of the current infrastructure with respect to
104 the limited water availability throughout the year. The hydraulic model is simulated for a two-
105 week period, one week representing the rainy season and the other for the dry season. The specific
106 objectives of the water utility are divided into three groups: first is related to the DMA design
107 features (i.e. the number of DMAs, total water demand similarity in DMAs); second is the
108 hydraulic performance and operation (i.e. water age of WDS, pressure uniformity in DMAs and

109 the number of the operational changes between the two seasons) and third is the total annual cost
110 of rehabilitation. Pressure uniformity and water age are calculated for both seasons separately. A
111 summary of the constraints proposed by the water utility is as follows: (1) maximum and minimum
112 pressure heads equal to 60m and 15m, respectively for demand nodes inside DMAs, and positive
113 pressure heads for non-demand nodes; (2) the water level in tanks at the end of simulation must
114 be at least the same water level at the beginning of the simulation period; (3) tank volume can
115 change between 10 and 90 percent of the tank capacity; (4) each DMA is allowed to have a
116 maximum of two inlets controlled by Pressure Reducing Valves (PRVs); (5) pressure setting of all
117 valves used in the WDS can change only between the seasons; (6) addition of new tanks is allowed
118 only adjacent to existing tanks; (7) addition of hydraulic controls to the existing pump stations is
119 allowed but variable speed pumps or time controlled pumps are not allowed; (8) the number of
120 DMAs must be at least 15 and the ideal number is also 15. The replacement of pipes is restricted
121 to pipes with diameters larger than 152mm. Further details of problem description can be found
122 on the BWNDMA website (Saldarriaga 2016) .

123

124 **Methodology**

125 A framework is proposed for optimal operation of WDS with new DMA configuration,
126 which consists of three sequential main phases: preliminary analysis, DMA configuration and
127 optimal operation. These phases are made up of 11 steps as shown in Fig. 1. The first phase
128 generally assists the designer in identifying key characteristics of the system and potential
129 bottlenecks and critical parts. Configuration of DMAs is conducted through four sequential steps
130 in the second phase. In the third phase, first a feasible hydraulic model is identified and then
131 optimal operation of WDS is generated. The software tools used in this paper are Gephi (2014) for

132 creation of DMAs and EPANET (Rossman 2000) for hydraulic and water quality model
133 simulation. The single objective and NSGA-II multi-objective evolutionary algorithms (Deb et al,
134 2002) are also used for optimisation of WDS operation. The details of each step are provided in
135 the following.

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

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

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

$$160 \quad Q = \frac{1}{2m} \sum_{v\omega} \left[A_{v\omega} - \frac{k_v k_\omega}{2m} \right] \delta(c_v, c_\omega) \quad (1)$$

161 where $A_{v\omega}$ = an element of the adjacency matrix of the network; $k_v = \sum_{\omega} A_{v\omega}$ = the sum
 162 of the number of edges connected to vertex v ; c_v = the cluster to which vertex v belongs,

163 $\delta(c_v, c_\omega) = 1$ if $c_v = c_\omega$, otherwise it is 0 and $m = \frac{1}{2} \sum_{v\omega} A_{v\omega}$ = the number of edges in

164 the graph. The required number of clusters is provided in Gephi by the "Resolution" parameter in
 165 the modularity settings. The default value of "Resolution" is set to one, higher values lead to fewer
 166 clusters and vice versa. As the level of decomposition is case-specific for any WDS analysis, an
 167 appropriate level of decomposition, e.g. number of clusters, could be discovered based on trial and
 168 error approach using different resolution values. Also, note that PRVs and gate valves which can
 169 be used for entrance and exit points of DMAs are defined close to the end of pipes (i.e. nodal
 170 points) in the EPANET model as specified by the water utility. The technique suggested by
 171 Giustolisi and Ridolfi (2014) is also an alternative approach which can be applied for dividing the
 172 WDS into segments based on the attributes of nodes and pipes.

173 **Step 3:** This step identifies transmission/trunk mains by which DMAs are isolated from each
 174 other. In other words, the DMAs are connected to the system through the trunk mains. In this study,

175 trunk mains are defined based on three key features: direct link between a water source and a tank,
176 flow rate capacity and pipe size. Thus, all ordinary pipes in the boundary of a DMA connected to
177 other DMAs are closed. This step also aims to maximise the number of DMAs fed directly by
178 water sources through trunk mains and hence minimise the number of DMAs fed from other DMAs
179 (Ferrari et al. 2014).

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

191 **Step 5:** Based on the constraint of maximum two PRVs at each DMA inlet (BWNDMA
192 2016), potential locations for installing PRVs at DMA boundaries are specified in this step. As
193 each DMA can be fed by either a trunk main or a neighbourhood DMA, the potential PRV locations
194 are either on the pipes connecting DMAs to trunk mains or pipes between neighbouring DMAs.
195 The potential PRV locations are considered as decision variables in the optimisation problems in
196 the next phase. The pipes which are not selected by the optimisation problems as optimum location
197 for PRVs at DMA boundaries will be closed.

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

$$202 \quad \text{Min}(F_1) = \sum_{j=1}^M \sum_{i=1}^N |P_{ij}| \quad \text{if } P_{ij} < 0 \quad (2)$$

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

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

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

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

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

$$238 \quad \text{Min } f_1 = PU_{net} = \sum_{k=1}^{M_s} \sum_{j=1}^{M_{time}} \left[\frac{1}{N_{junc}} \sum_{i=1}^{N_{junc}} \frac{(P_{ijk} - P_{min})}{P_{min}} + \frac{\sqrt{\sum_{i=1}^{N_{junc}} \frac{(P_{ijk} - P_{avjk})^2}{N}}}{P_{avjk}} \right] \quad (3)$$

$$239 \quad P_{avj} = \frac{\sum_{i=1}^{N_{junc}} P_{ij}}{N_{junc}} \quad (4)$$

$$240 \quad Min f_2 = WA_{net} = \frac{\sum_{k=1}^{M_s} \sum_{i=1}^{N_{junc}} \sum_{j=1}^{M_{time}} k_{ijk} Q_{dem,ijk} (WA_{ijk} - WA_{th})}{\sum_{i=1}^{N_{junc}} \sum_{j=1}^{M_{time}} Q_{dem,ijk}} \quad (5)$$

$$241 \quad k_{ijk} = \begin{cases} 1, & WA_{ijk} \geq WA_{th} \\ 0, & WA_{ijk} < WA_{th} \end{cases} \quad (6)$$

$$242 \quad Min f_3 = CC_{net} = \sum_{i=1}^{NN} C_i L_i + \sum_{j=1}^{MM} K_j + \sum_{k=1}^{OO} V_k \quad (7)$$

243 where PU_{net} = the system pressure uniformity; P_{ijk} = the pressure at junction i , time j and season k ; P_{min}
244 = the minimum allowable pressure (15 m); P_{avjk} = the average pressure in the system at time j and season
245 k ; WA_{ijk} = the water age at junction i , time j and season k in hour; WA_{net} = the system water age in hour;
246 WA_{th} = the water age threshold (60 hours); $Q_{dem,ijk}$ = the demand at junction i , time j and season k ; CC_{net}
247 = the total system capital cost; C_i , L_i = the unit cost and total length of rehabilitated pipe i ; K_j = the cost
248 of installed valve j ; V_k = the cost of added volume in tank k ; NN , MM and OO = total number of
249 rehabilitated pipes, installed valves and tanks with added volume in both seasons; N_{junc} = the number
250 of system junctions; M_{time} = the number of simulation time steps (equal to 168 one-hour steps as the
251 extended period simulation time of one week) and M_s = the number of seasons (i.e. 2 including rainy
252 and dry). Note that pressure uniformity and water age indicators defined above are evaluated for the
253 junctions with non-zero water demand. Also, the cost function (f_3) includes annual capital investment

254 for rehabilitation only (i.e. not operational costs such as energy costs) as set by the water utility
255 (BWNDMA 2016).

256

257 **Results and discussion**

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

266 Graph theory method is used to generate 15 DMAs in the WDS in step 2 to comply with the
267 desirable number of DMAs specified by the water utility (BWNDMA 2016). A total of 968 pipes
268 are identified in step 3 as transmission/trunk mains, with a pipe diameter ranging between 300 and
269 1524 mm, which are around 9% of the total WDS pipes. Modification of the DMA configuration
270 is made for 10 DMAs in step 4 based on nodal elevations and layout of trunk mains to improve
271 the similarity of total water demand of DMAs. More specifically, some DMAs with large total
272 water demand are split into several new DMAs while some DMAs with small total water demand
273 are merged together (see some examples in Fig. 2). The final configuration of DMAs is obtained
274 in step 4 (Fig. 3) with the main characteristics given in Table 2. As can be seen, three distinct
275 groups of similar total water demand can be observed including 6 DMAs (i.e. 1, 2, 3, 4, 5 and 6)

276 with a weekly average demand of around 85,000 m³, 2 DMAs (i.e. 7 and 9) with slightly larger
277 demand (around 97,000 m³) and 7 DMAs (i.e. 8, 10, 11, 12, 13, 14 and 15) with relatively smaller
278 demand (around 70,000 m³). Further investigation of the large total water demand in DMAs 7 and
279 9 revealed that there are a few small branches that cannot be separated and combined with other
280 DMAs due to their specific geographical locations. A total of 135 potential locations are also
281 identified in step 5 as potential locations for PRV installation at the entrance/boundary points of
282 DMAs.

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

292 Fig. 4 shows the Pareto fronts of optimal solutions obtained from the multi-objective
293 optimisation runs described in step 10. The trade-offs between the three objective functions
294 indicate a strong correlation between the annual capital cost and the pressure uniformity (PU)
295 indicator. In other words, this shows that the PU can be improved in the system through the capital
296 investment in pipeline rehabilitation, increasing number of DMAs (Salomons, et al. 2017) and
297 upgrading tanks. More specifically, pipeline rehabilitation can result in decreasing pressure head
298 losses and hence pressure differences in DMAs. Upgrading tanks can also prevent from pressure

299 differences during the simulation especially during peak times. Any solution in these Pareto fronts
300 can be chosen and introduced as the selected optimal solution. The selected solution is better than
301 80 percent of solutions on Pareto front with respect to the cost and is the closest to the best values
302 of the other two objective functions (This solution is shown by a black square in the Pareto fronts
303 of the figure).

304 A feasible solution is generated following optimisation model runs in steps 6 and 8. In step
305 10, a solution with maximum two PRVs at entry points of each DMA is selected from Pareto fronts
306 generated by the multiobjective optimisation model. Table 2 provides characteristics of the optimal
307 solution obtained in these steps for both seasons. The connections between the DMAs and trunk
308 mains during the rainy season shows that 7 DMAs are fully independent (i.e. supplied directly
309 from the water sources through trunk mains only), 4 DMAs are partially independent (i.e. supplied
310 from both trunk mains and other DMAs) and 4 DMAs are fed by adjacent DMAs. During the dry
311 season, the only switch of water supply happens in DMA1 where its water sources change from
312 partially independent to fully independent (i.e. both connections are fed through trunk mains). This
313 shows that the majority of the DMAs in the solution take the advantage of direct supply from the
314 trunk mains as desired by water utilities (Ferrari et al. 2014).

315 Table 3 also shows the characteristics of the 17 tanks in the selected solution. Out of 17
316 tanks, only 8 tanks need to be upgraded with added volume ranging between 1000 and 5750 m³.
317 The optimal solution shows that the tanks have a key role in providing the balance between diurnal
318 water demand and supply in the WDS. Out of 17 existing tanks, 7 tanks remain inactive always
319 and for 3 more (Tanks 1, 11 and 17) become active only during the dry season due to limited water
320 available in water sources during peak times. Also, upgrading four tanks with a large capacity
321 (Tanks 1, 12, 14 and 16) seems to be quite important as each of them supplies water for more than

322 one DMA directly or indirectly. This can be seen by comparing the associated DMAs of these
323 tanks (DMAs 1, 5, 8, 9, 10, 11 and 12) in Table 3 and their connection with other DMAs (DMAs
324 2, 4, 6 and 13) in Table 2. Furthermore, operation of some tanks (e.g. Tanks 5 and 11) have had a
325 key role in eliminating negative pressures in the relevant DMAs during peak times in the dry
326 season. More specifically, Tanks 5 and 11 are at the highest elevation compared to other points in
327 the related DMAs (i.e. 14 and 15) and elevation of almost all other tanks in the system. In addition,
328 most of the topography of the DMAs is flat, except DMAs 1, 14 and 15 which have relatively high
329 elevation differences. In particular, DMA 14 has the highest point in the system and hence
330 providing a balance of water supply between lowest and highest elevation points is difficult.
331 Therefore, tanks 5 and 11 can guarantee that the highest demand points are supplied with sufficient
332 pressure during peak times.

333 Due to the changes in both flow capacity and type of water sources between the two
334 seasons, FCV settings located at the entrance of tanks have vital role in providing the balance
335 between water demand and supply during peak times. For instance, Tank 14 is fed by only one
336 water source (WTP1) with a maximum flow of 800 l/s during the rainy season but the water sources
337 feeding the tank changes in the dry season to two sources (WTP1 and a pump station of SP1) with
338 a total of maximum flow of 626 l/s. The results show that FCV settings in Tank 14 is reduced from
339 233.2 l/s in the rainy season to 161 l/s in the dry season. This can be due to the fact that the network
340 (i.e. related DMAs) can be directly fed by Mohan pump station (SP1) which can also provide
341 sufficient pressure head. However, the only water source available in the rainy season (i.e. WTP1)
342 is unable to provide sufficient pressure head for directly feeding the network during peak times
343 and hence Tank 15 needs to be used more by filling more water through a larger FCV setting.
344 Similarly, the settings of other FCVs should be more limited due mainly to the lack of available

345 water and benefiting from the two pump stations (SP1 and SP2) working only during the dry
346 season.

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

365 **Conclusions**

366 A new framework was presented, comprising three phases based on preliminary analysis, a
367 graph theory and single-and multi-objective optimisation approaches, to identify a desirable DMA

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

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

- 390
- Due to the major changes in the capacity and the type of water supply sources between the
391 two seasons, the optimal operation of the WDS uses more capacity of the storage tanks in
392 the rainy season whilst the storage tanks is used less in the dry season and it is more
393 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
394 values in the dry season as water demands partially supplied directly by pump stations.
395 This results in lower water age in the dry season than that in the rainy season when the
396 water is more stored in the tanks.
397

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

407 **Acknowledgements**

408 The work reported is supported by the UK Engineering & Physical Sciences Research Council
409 (EPSRC) project Safe & SuRe (EP/K006924/1).

- Ali, M. (2015). Knowledge-Based Optimization Model for Control Valve Locations in Water Distribution Networks. *Journal of Water Resources Planning and Management*, 141(1), p.04014048.
- Alvisi, S. and Franchini, M., (2014). “A heuristic procedure for the automatic creation of district metered areas in water distribution systems.” *Urban Water J.*, 11(2), 137-159.
- Araujo, L. S., Ramos, H., and Coelho, S. T. (2006). “Pressure control for leakage minimisation in water distribution systems management.” *Water Resour. Manage.*, 20(1), 133–149.
- BWNDMA (2016). “Battle of water Networks District Metered Area”. [Wdsa2016.uniandes.edu.co](http://wdsa2016.uniandes.edu.co). Available at: <https://wdsa2016.uniandes.edu.co/index.php/battle-of-water-networks> [Accessed 22 Jan. 2017].
- Clauset A., Newman M. E. J. and Moore, C. (2004). “Finding community structure in very large networks.” *Phys. Rev. E: Stat. Phys. Plasmas Fluids Relat. Interdiscip. Top.*, 70(066111).
- Deb, K., Pratap, A., Agarwal, S., and Meyarivan, T. A. M. T. (2002). “A fast and elitist multiobjective genetic algorithm: NSGA-II.” *IEEE transactions on evolutionary computation*, 6(2), 182-197.
- Di Nardo, A, Di Natale, M, Guida, M, Musmarra, D (2013) Water Network Protection from Intentional Contamination by Sectorization, *Water Resour. Manage.*, 27(6), 1837-1850.
- Diao, K., Farmani, R., Fu, G., Astaraiie-Imani, M., Ward, S. and Butler, D., (2014). “Clustering analysis of water distribution systems: identifying critical components and community impacts.” *Water Sci. Technol.*, 70(11), 1764-1773.
- Farmani, R., Walters, G. A., and Savic, D. A. (2005). “Trade-off between total cost and reliability for any town water distribution network.” *J. Water Resour. Plann. Manage.*, 10.1061/(ASCE)0733-9496(2005) 131:3(161), 161–171.

- Farmani, R., Walters, G., and Savic D. A. (2006) “Evolutionary multi-objective optimisation of the design and operation of water distribution network: total cost vs. reliability vs. water quality”, *J. of Hydroinformatics*, 8(3), 165-179.
- Ferrari, G., Savic, D. and Becciu, G., (2014). “Graph-theoretic approach and sound engineering principles for design of district metered areas.” *J. Water Resour. Plann. Manage.*, 140(12), p.04014036.
- Gephi.org, (2014). Gephi, an open source graph visualization and manipulation software. [online] Available at: <https://gephi.org/> [Accessed 22 Jun. 2014].
- Giustolisi, O. and Ridolfi, L. (2014). New Modularity-Based Approach to Segmentation of Water Distribution Networks. *J. Hydraul. Eng.*, 140(10), p.04014049.
- Giustolisi, O., Simone, A., & Ridolfi, L. (2017). Network structure classification and features of water distribution systems. *Water Resour. Res.*, 53(4), 3407-3423.
- Gomes, R, Sá Marques, A, Sousa, J (2012) Decision support system to divide a large network into suitable District Metered Areas. *Water Science & Technology*, IWA Publishing, 65(9), 1667-1675.
- Kapelan, Z.S., Savic, D.A. and Walters, G.A., 2005. Multiobjective design of water distribution systems under uncertainty. *Water Resour. Res.*, 41(11).
- Laucelli, D., Simone, A., Berardi, L. and Giustolisi, O. (2017). Optimal Design of District Metering Areas for the Reduction of Leakages. *J. Water Resour. Plann. Manage.*, p.04017017.
- Mahdavi, M.M., Hosseini K., Behzadian K., Ardehsir A., and Jaliliani, F. (2010). “Leakage control in water distribution networks by using optimal pressure management: A case study.” *Water Distribution Systems Analysis conference*, 1110-1123.

- Marchi, A., Salomons, E., Ostfeld, A., Kapelan, Z., Simpson, A.R., Zecchin, A.C., Maier, H.R., Wu, Z.Y., Elsayed, S.M., Song, Y. and Walski, T., (2013). “Battle of the water networks II.” *J. Water Resour. Plann. Manage.*, 140(7), p.04014009.
- Matthews, J.C., Piratla, K. and Matthews, E., (2014). “Disaster resilience of drinking water infrastructure systems to multiple hazards.” In *Structures Congress*, 2312-2317.
- Muhammed, K.A., Farmani, R., Behzadian, K., Diao, K. and Butler D. (2017) “Optimal Rehabilitation of Water Distribution Systems using a Cluster-based Technique.” *J. Water Resour. Plann. Manage.* 10.1061/(ASCE)WR.1943-5452.0000770.
- Nafi, A., and Kleiner, Y. (2009). “Scheduling renewal of water pipes while considering adjacency of infrastructure works and economies of scale.” *J. Water Resour. Plann. Manage.*, 136(5), 519-530.
- Newman, M. E. J. (2006). “Modularity and community structure in networks”. *PNAS*, 103(23), 8577-8582.
- Ostfeld, A., Oliker, N. and Salomons, E., (2013). “Multiobjective optimization for least cost design and resiliency of water distribution systems.” *J. Water Resour. Plann. Manage.*, 140(12), p.04014037.
- Rahmani, F., Ardeshir, A., Behzadian, K. and Jalilsani, F., (2014). “Optimal Rehabilitation Strategy in Water Distribution Systems Considering Reduction in Greenhouse Gas Emissions.” HIC2014–11th International Conference on Hydroinformatics, New York, USA.
- Rahmani, F., Behzadian, K. and Ardeshir, A., (2015). “Rehabilitation of a water distribution system using sequential multiobjective optimization models.” *J. Water Resour. Plann. Manage.*, 142(5), p.C4015003.

- Rossman, L. A. (2000). “EPANET2.” EPA/600/R-00/057, Water Supply and Water Resources Division, National Risk Management Research Laboratory, Office of Research and Development, U.S. EPA, Cincinnati.
- Saldarriaga, J. (2016). “Battle of water networks DMAs (BWNDMA) — Problem description and rules. ” ([https://wdsa2016.uniandes.edu.co /index.php/battle-of-water-networks](https://wdsa2016.uniandes.edu.co/index.php/battle-of-water-networks)) (August. 16, 2017).
- Saldarriaga, J., Páez, D., Bohórquez, J., Páez, N., París, J., Rincón, D., Salcedo, C. and Vallejo, D. (2016). Rehabilitation and Leakage Reduction on C-Town Using Hydraulic Criteria. *J. Water Resour. Plann. Manage.*, 142(5), p.C4015013.
- Salomons, E., Skulovich, O. and Ostfeld, A. (2017). Battle of Water Networks DMAs: Multistage Design Approach. *Journal of Water Resources Planning and Management*, 143(10), p.04017059.
- Scarpa, F., Lobba, A. and Becciu, G. (2016). Elementary DMA Design of Looped Water Distribution Networks with Multiple Sources. *J. Water Resour. Plann. Manage.*, 142(6), p.04016011.
- Sedki, A. and Ouazar, D., (2012). “Hybrid particle swarm optimization and differential evolution for optimal design of water distribution systems.” *Adv. Eng. Inf.*, 26(3), 582-591.
- Shafiee, M., Berglund, A., Berglund, E., Brill, E. and Mahinthakumar, G. (2016). Parallel Evolutionary Algorithm for Designing Water Distribution Networks to Minimize Background Leakage. *J. Water Resour. Plann. Manage.*, 142(5), p.C4015007.
- Thornton, J., Sturm, R. and Kunkel, G., 2008. Water loss control. McGraw Hill Professional.
- Vassiljev, A. and Puust, R. (2016). Decreasing Leakage and Operational Cost for BBLAWN. *J. Water Resour. Plann. Manage.*, 142(5), p.C4015011.
- UN, 2005.