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

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20   **Optimal Operation of Water Distribution Systems using a Graph**

21   **Theory-Based Configuration of District Metered Areas**

22   **Abstract**

23   Optimal operation of large water distribution systems (WDS) has always been a tedious task

24   especially when combined with determination of district metered areas (DMAs). This paper

25   presents a novel framework based on graph theory and optimisation models to design DMA

26   configuration and identify optimal operation of large WDS for both dry and rainy seasons. The

27   methodology comprise three main phases of preliminary analysis, DMA configuration and optimal

28   operation. The preliminary analysis assists in identifying key features and potential bottlenecks in

29   the WDS and hence narrow down the large number of decision variables. The second phase

30   employs a graph theory approach to specify DMAs and adjust their configuration based on

31   similarity of total water demands and pressure uniformity in DMAs. Third phase uses several

32   consecutive single-objective and multi-objective optimisation models. The decision variables are

33   pipe rehabilitation, tank upgrade, location of valves and pipes closure, and valve settings for each

34   DMA. The objective functions are to minimise total annual cost of rehabilitation, water age and

35   pressure uniformity. The proposed methodology is demonstrated through its application to large

36   real-world WDS of E-Town. The results show that the proposed methodology can determine a

37   desirable DMA configuration mainly supplied directly by trunk mains.

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

39

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

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 at 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

4

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

5

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

161   where $A_{\upsilon\omega}$ = an element of the adjacency matrix of the network; $k_\upsilon = \Sigma_\omega A_{\upsilon\omega}$ = the sum

162 of the number of edges connected to vertex υ; c_υ = the cluster to which vertex υ belongs,

163 $\delta(c_\upsilon, c_\omega) = 1$ 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

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

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

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

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

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

242
$$Min \; f_3 = CC_{net} = \sum\limits_{i=1}^{NN} C_i L_i + \sum\limits_{j=1}^{MM} K_j + \sum\limits_{k=1}^{OO} V_k$$
(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

12

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)

13

276 with a weekly average demand of around 85,000 m^3, 2 DMAs (i.e. 7 and 9) with slightly larger

277 demand (around 97,000 m^3) and 7 DMAs (i.e. 8, 10, 11, 12, 13, 14 and 15) with relatively smaller

278 demand (around 70,000 m^3). 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

14

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 form 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 m^3.

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

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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.

394   • As a result of the above changes, settings of FCVs in most of the tanks is reduced to lower

395   values in the dry season as water demands partially supplied directly by pump stations.

396   This results in lower water age in the dry season than that in the rainy season when the

397   water is more stored in the tanks.

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.

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