PRESSURE MANAGEMENT IN WATER DISTRIBUTION NETWORK BY MULTI-OBJECTIVE GENETIC ALGORITHM

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ABSTRACT

This research paper exhibits a methodology for optimal pressure regulation in water distribution systems by determination of the number, locations, and opening size percentages of Throttle Control Valves (TCVs). This methodology is considered an efficient, reliable and applicable for any water distribution network, which enables the pumps to operate in pre-specified ranges and on the other hand, reduces undesirable excessive nodal pressure heads satisfying all hydraulic constraints. The objective of the study is to minimize both the number of TCVs and the root mean square errors between nodal pressure heads and minimum allowable heads. Multi-objective genetic algorithm technique is linked with both a new technique used for simulation of pipe network in steady-state conditions and a new approach used for determination of a compromise solution from a set of Pareto optimal solutions. The great advantage of the proposed methodology is to select one Pareto optimal solution for the multi-objective problem instead of several trade-off alternatives. At the same time, in one run, the necessary data of TCVs for the obtained Pareto optimal solution can be determined. A computer code, called MO-OPTIM, has been written initially to apply the mathematical principles of the problem. This code was applied in a real water distribution system of Damnhour city, Egypt. Finally, a comparison is carried out between leakage volume in two cases of controlled and uncontrolled pressure heads.

Keywords: Multi-objective optimization; Genetic algorithm; Pressure regulation; Leakage reduction; compromise solution; Throttle control valves.

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1 INTRODUCTION

Leakage in water supply networks represents a large percentage of the total supplied water, depending on the age and deterioration of the system (Reis et al., 1997). There is a difference between total water loss and leakage. Total water loss is the difference between the total supplied water and the amount of consumed water. Leakage is one of the components of water loss and comprises physical losses from pipes, joints, and fittings and also overflows from service reservoirs (Awad, 2005). It is desirable to supply consumers with water at appropriate pressure as the excess of pressure may cause water leakage from pipes. In some aging and deteriorating urban distribution networks, leakage value is high up to 50 percent from the total supplied water (Jowitt and Xu, 1990), for this reason, network pressure heads have to be regulated to an adequate level. Minimization of leakage/ excessive pressure is performed using a system of the pressure reducing valves or throttle control valves or flow control valves. These valves are variable closure devices that reduce the capacity of the pipe in which they are located and increase the pressure loss across the pipe.

Jowitt and Xu (1990) studied the problem of minimization of leakage in water distribution networks via determination of flow control valve settings for a given number of control valves. A linear programming was used to formulate equations and consequently minimization of leakage. They concluded that leakage reduction during the peak demand period is marginal whereas became maximum during the night when consumers demands are lower, and system pressure tends to be higher. Reis et al. (1997) used a genetic algorithm technique to calculate the optimal location for a given number of control valves as well as their settings for maximization of leakage reduction at given nodal demands and reservoir levels. They concluded that leakage control could be obtained with the smallest number of valves when they are located optimally in the network. Vairavamoorthy and lumbers (1998) developed an optimization method to minimize leakage in water distribution systems by finding the optimal valve settings for a given number of valves. This problem was solved by using Sequence of Quadratic Programming (SQP) method, which is considered as an approximation for the original problem, to generate at each step a search direction that is used to update the solution vector. Tang et al. (2000) examined a software model consists of Genetic Algorithm Processor (GAP) and a transient pipe network analysis to calculate the location and size of leaks in a blind test. The location and size of leaks were experimentally calculated at the leak test system in the hydraulic laboratory of the University of Perugia, Italy. Then, the model was applied to the test system to determine the location and size of leaks. Awad (2005) presented a modeling technique and control scheme for minimizing leakage in water distribution networks by regulating the pressure in all the network nodes between the upper and the lower limits and/or as near as possible to a target value. Araujo et al. (2006) used a genetic algorithm for optimization the number of valves and their locations, as well as valves openings with the objective of minimization of pressures and consequently leakage reduction. They concluded that the higher number of valves does not give the best solution. Nicolini and Zovatto (2009) presented a methodology for optimal pressure management and consequently leakage reduction from water distribution systems containing pressure reducing valves. The multi-objective genetic algorithm was used to determine the number, locations, and settings of valves. The objectives of the study were to minimize both the number of valves and total leakage in the system after satisfying the required head at each node. Tricarico et al. (2014) presented multi-objective optimization methodology to minimize leakage and in the same time to minimize the difference between operational pumping costs and income generated through energy recovery by strategically locating in the network pumps operating as turbines using pressure-driven analysis. The presented methodology is showed its efficiency compared with using pressure reducing valves (PRVs). Samir et al. (2017) using (PRVs) for pressure controlling and minimization of leakage in water distribution networks. The application is carried out on a district metered area in Alexandria, Egypt. Several scenarios were presented and the results showed that, the leakage was dropped by 37% for the best scenario. Gupta et al. (2017) presented a methodology using PRVs for leakage reduction in water distribution networks. The suggested methodology was divided into two parts. The first part concerns with determination of reference pressure for the calibration of the number of PRVs required along with their location. In the second part, they used NSGA-II software to determine the optimized pressure of the valve (first objective) and to minimize the leakage rate (second objective). The results showed that, the leakage rate was reduced by 6.15 l/s (20.82%) after using the proposed algorithm in a benchmark WDS, considering all the load condition.

The primary purpose of this research paper is to suggest a methodology for determination of number, locations, opening size percentages of TCVs in water distribution systems, with the objective of minimizing undesirable excessive nodal pressure heads (and consequently leakage reduction) and enables the pumps to operate in pre-specified ranges.

2 MATHEMATICAL FORMULATION

This section presents a mathematical formulation for regulation of the pressure in all network nodes between upper and lower limits, for leakage minimization in water distribution networks. Pressure regulation is performed by predicting the optimal locations, number as well as opening sizes of the (TCVs) for a network having no valves. Then, the problem under consideration has to be formulated as a nonlinear multi-objective optimization one consisting of two objective functions subjected to both linear and nonlinear equality and inequality constraints.

2.1 Objective Functions

The first objective function of the optimal pressure regulation is the minimization of the Root Mean Square Error (RMSE) between nodal heads and minimum allowable head as follows (Awad, 2005):

$$Obj_1 = \left[\frac{1}{n} \sum_{j=1}^{n} \left(h_j - h_j^T \right)^2 \right]^{1/2} \tag{1}$$

in which, h_j = head at node j, h_j^T = required target head at the same node, and n = number of nodes in the network.

The second objective function is to minimize the number of TCVs as follows:

$$Obj_2 = N_V \tag{2}$$

in which, N_V = number of valves.

2.2 Constraints

Constraints can be classified into three groups, implicit bound constraints, explicit variable constraints and implicit system constraints.

The implicit bound constraints*:* these bound constraints include restrictions on nodal heads (*h*), pipe velocity (*V*), and pump head limits (h_p). Mathematically, these constraints can be expressed as follows:

$$h_j \geq H_{min} \quad , \quad j = 1 , \ldots\ldots\ldots, n \tag{3}$$

$$V_k \leq V_{max} \quad , \quad k = 1 , \ldots\ldots\ldots, p \tag{4}$$

$$h_{Pi\,min} \leq h_{pi} \leq h_{Pi\,max} \quad , \quad i = 1 , \ldots\ldots\ldots, NP \tag{5}$$

in which, h_j = pressure head at node j; H_{min} = minimum allowable head*;* n = number of nodes through the network; V_k = velocity of flow through pipe k; V_{max} = maximum allowable flow velocity; p = number of pipes through the network; h_{pi} = head produced by a pump i; $h_{Pi\,min}$ and $h_{Pi\,max}$ = minimum and maximum allowable pump head for each pump i; and NP = number pumps through the network.

The explicit variable constraints*:* these variable constraints can be used to set limits on opening size percentages of the TCVs. Mathematically, these constraints can be expressed as follows:

$$0 \leq \theta_v \leq 100 \quad , \quad N_V = 1, \ldots\ldots\ldots, V \tag{6}$$

in which, θ_v = percentage of TCV opening size, and N_V = number of TCVs.

The implicit system constraints*:* these system constraints include nodal conservation of mass and conservation of energy. Mathematically, these constraints can be expressed as follows:

1. Nodal conservation of mass: inflow and outflow must be balanced at each junction node as follows:

$$\sum Q_{in} - \sum Q_{out} = Q_e \tag{7}$$

for each junction node (other than the source, i.e. excluding reservoir and tanks) in which, Q_{in} = flow into the junction, Q_{out} = flow out of the junction, and Q_e = external inflow or demand at the junction node.

2. Conservation of energy: head loss around a closed loop must equal zero or pump energy head if there is a pump.

$$\sum h_f = zero \quad \textit{(around each loop in case of there is no pump)} \tag{8}$$

$$\sum h_f = E_p \quad \textit{(if there is a pump)} \tag{9}$$

in which, h_f = head loss due to friction in a pipe, and E_p = the energy supplied by a pump.

2.3 Decision Variables

Decision variables/unknowns, when the network does not contain valves, are number of valves as well as their locations/ pipe numbers and setting of each. The number of genes in every chromosome is an equal number of pipes in the studied network. The first gene corresponds to the first pipe and the second gene corresponds to the second pipe and so on. To determine the decision variables in each chromosome generated random number value between 0 and 1 for each pipe in the network/gene. If this value is less than 0.5 then the pipe under consideration does not contain a valve and the corresponding gene has value equal (0). Otherwise, the pipe contains a valve. In case of there is a valve repeatedly generated random number value to obtain the valve opening percentage value between 0% and 100 %. For each gene has a value other than zero then, the corresponding pipe have a valve of the opening has the same value.

3 SOLUTION METHODOLOGY

A new technique used for simulation of pipe network in steady-state conditions (El-Ghandour, 2010), multi-objective genetic algorithm technique and a new approach used for determination of a compromise solution from a set of Pareto optimal solutions (Grierson, 2008) are used as follows.

3.1 Pipe Network Hydraulic Analysis for Steady-State Conditions

Both Linear Theory Method (LTM) (Larock et al., 2000) and Extended Linear Graph Theory (ELGT) (Gupta and Prasad, 2000) are linked to getting a new technique which could be used for the analysis of pipe networks. This technique differs from other linear theory methods in the system formation of linear equations and solution procedures. The solution algorithm used in this technique is independent on initial pipe flows estimation, where a power law equation is used to update the pipe flows in successive iterations. The proposed method has been extended to deal with complex systems including control devices such as pumps, Pressure Reducing Valves (PRVs), Pressure Sustaining Valves (PSVs), and Check Valves (CVs).

3.2 Genetic Algorithm Technique

This technique is a search method that uses the mechanisms of natural selection to search through decision space to optimal solutions. GA has shown to be a valuable tool for solving complex optimization problems in a broad spectrum of fields. The GA based "solution method can generate both convex and non-convex points of the trade-off surface, and accommodate non-linearities within the multiple objective functions (El-Ghandour, 2010). GA consists of three basic operations as given by Goldberg (1989): (1) selection, (2) crossover, and (3) mutation. In using GA, several chromosomes which represent different sets are formed randomly. These chromosomes are evaluated on their performance/fitness with respect to some objective functions.

A brief description of the multi-objective genetic algorithm solution can be described in the following steps:

1. Generation of the initial population: this step generates an initial population of chromosomes randomly, ranges from 100 to 200, and puts them in the father pool. Every chromosome within the created population consists of the number of genes equal to the number of unknown variables. In the problem under consideration, unknown variables are a number, locations, and opening size percentages of TCVs.

2. Hydraulic analysis of each network: using the data included in every chromosome located within the father pool, the hydraulic analysis is applied using the new technique mentioned in the previous sub-section to calculate the nodal pressure heads and the flow rates through pipes.

3. Computation of objective function: each objective function, Eqs. (1) and (2), is computed separately for each chromosome.

4. Computation of penalty: for any constraint, which does not satisfy the required limits, the corresponding penalty has to be assigned.

5. Computation of total objective function: each penalty, computed in step (4), on any constraint which has not been satisfied is added to the related objective function, computed in step (3), to obtain the total objective functions corresponding to each chromosome.

6. Computation of fitness: to compute the fitness of each chromosome, a layer classification technique is used whereby the population is incrementally sorting using Pareto dominance (Ngatchou et al., 2005). The following steps exhibit the method of calculation of the fitness for each chromosome (Liu and Hammad, 1997):

- All chromosomes in the current population are compared, according to their total objective functions, to determine the Pareto optimal set of this population and assign a rank of 1 for this set. A chromosome belongs to the Pareto set if there is no other chromosome that can improve at least one of the objectives without degradation of any other objective (Ngatchou et al., 2005). In other words, a solution/chromosome is called Pareto optimal solution if it beats all other solution at least in one criterion/objective.
- The set of chromosomes having rank 1 is set apart, and the remaining chromosomes are compared to select a new non-dominated/Pareto set with a rank of 2.
- This process continues until the entire population is ranked.
- The fitness function value of each chromosome is assigned according to its rank, using the following equation (Liu and Hammad, 1997):

$$F_i = 1/rank_i \tag{10}$$

in which, F_i and $rank_i$ are the fitness and the rank number of individual i respectively.

7. Aggregation of Pareto solution: for each generation, a set of Pareto solutions that has the rank of 1 is copied in a separated pool called Pareto pool.

8. Replacement strategy: replacement strategy is performed by replacing the set of weakest strings that has greater rank from the children pool with the fittest one from the father pool that has lesser rank. The replacement of each chromosome is performed in case of there is no identical chromosome.

9. Generation of a new population: this step uses the selection, crossover, and mutation operators as follows:

- Selection selects two chromosomes randomly according to its fitness, Eq. (10), using the roulette wheel method (Goldberg 1989).
- Crossover generates a uniform random number between 0 and 1 and compares this number with crossover ratio. If it is smaller than the crossover ratio, the crossover on the two chromosomes is applied to create one child chromosome using a uniform crossover between different genes. Otherwise the fittest chromosome is taken and put in the children pool.

- Mutation: for each gene within the children chromosome iteratively generates a uniform random number between 0 and 1, if it is smaller than the mutation ratio, the mutation has to be applied to this gene. This mutation operates randomly by a new value for this gene.
- The previous operators have to be repeated iteratively for each newly created chromosome in the children pool.

10. Production of successive generations: steps from 2 to 9 are repeated to generate successive generations.

11. Termination of the code: the code is terminated either when the number of generation is reached to the maximum generation number, or when the length between the two Pareto optimal sets in the Pareto pool is less than the allowable tolerance and repeated to 30 successive generations. The length between two sets in successive generations is equal to the summation of the minimum distance between each solution in the current set and all solutions in the previous one.

12. Results of the code: results of code contain the Pareto optimal solution set from all solutions located in the Pareto pool that aggregated in all generations.

3.3 Pareto-Compromise Solution

The objective of any multi-objective optimization is to find the set of acceptable solutions and present them to decision makers. A new technique based on a theorem proposed by Grierson (2008) to choose a compromise solution from a set of Pareto optimal solutions for which the competing criteria/objectives are mutually satisfied in a Pareto optimal sense. This technique is called Multi-Criteria Decision Making (MCDM) strategy.

The theorem is called PEG which states: "from among the theoretically infinite number of feasible designs forming the Pareto front for a design governed by n independent criteria f_i (i=1, n), there exists a unique Pareto-compromise design f_i^0 (i=1, n) that represents a mutually agreeable trade-off between all n criteria" (Grierson, 2008).

The aim of proposed multi-criteria decision-making strategy is to find *n* criteria values defining a unique Pareto-compromise design to be mutually agreeable for all $n \geq 2$ criteria, is referred as the PEG-MCDM procedure, and constitute from the following steps:

1 Determination of the multi-objective problem under consideration that may take the following form:

Minimize $\{f_1(z),........, f_n(z)\}$ subjected to $z \in \Omega$ (11)

in which, f_i *(i=1,.....n)* are the objective functions, expressed in terms of the design variable vector *z* in the feasible domain Ω for the n-dimensional criteria space.

2 Having the solution to the Pareto design optimization problem, Eq. (11), represented by the set of m – dimensional objective criteria vectors $f_i^*(i=1,....., N)$ defining the original Pareto data.

3 Identification of the extreme vector entries $f_i^{\max}$, $f_i^{\min}$ *(i=1,…., N)*.

4 Using the following equation to normalize the original Pareto data to find the *m*-dimensional vectors:

$$X_i = \frac{\left(f_i^* - f_i^{\min}\right)}{\left(f_i^{\max} - f_i^{\min}\right)}; (i=1,...., N) \qquad (12)$$

5 For N=2

a. set $X_1=x$, $X_2=y$

b. from the following equation, for $\delta x = \delta y = \sqrt{2} - 1$, find the shifted vectors x^*, y^*.

$$x^* = (x + \delta x)/(1 + \delta x) = [x^{*\min}, ..., x^{*\max}]^T = [1 - \sqrt{2}/2, ..., 1]^T \quad (13.a)$$

$$y^* = (y + \delta y)/(1 + \delta y) = [y^{*\max}, ..., y^{*\min}]^T = [1, ..., 1 - \sqrt{2}/2]^T \quad (13.b)$$

c. from the following equations find the redial shift Δr_0.

$$\Delta x_0 = \Delta y_0 = 0.5 - (x_j^* - x_{j+1}^*)(y_j^* - y_{j+1}^*)/(x_j^* + x_{j+1}^* + y_j^* + y_{j+1}^*) \quad (14)$$

where vector index j is such that $x_j^* / y_j^* \le 1$ while $x_{j+1}^* / y_{j+1}^* \ge 1$.

$$\Delta r_0 = \sqrt{2\Delta x_0} = \sqrt{2\Delta y_0} \quad (15)$$

The previous equations explain how to add several effects to the Pareto optimal solutions after normalization to take the shape of a quarter of a circle. The compromise solution exists in the quarter of a circle that has a coordinate of (0.5, 0.5). After determining the compromise solution, the several effects added to the original Pareto data are removed, and the position of compromise solution is determined. Usually, the compromise solution does not coincide with any existed solution of the original data, in this case, the nearest solution to the compromise one is taken.

6 For N > 2

a. from the following equations, the primary-aggregate vectors x_i and y_i, *(i=1,....N)* are assign:

$$x_i = [X_i^{\min}, ..., X_i^{\max}]^T = [0, ..., 1]^T; (i = 1, 2, ...N) \quad (16)$$

$$Y_i = \left(\sum_{k=1}^{n} x_k - x_i \right) \Big/ (n-1); (i = 1, 2, ..., N) \quad (17)$$

$$y_i = [X_i^{\max}, ..., X_i^{\min}]^T = [1, ..., 0]^T; (i = 1, 2, ...N) \quad (18)$$

b. from step 5(b), find the shifted vector x^*, y^* *(i=1,N)*.

c. from step 5(c), find the redial shifts Δr_i *(i=1,N)*.

7 From the PEG-function, find the objective criteria values $f_i^0 (i = 1, N)$ for the Pareto compromise design according to the following equation:

$$f_i^0 = f_i^{\max} - (f_i^{\max} - f_i^{\min})(\Delta r_i + \sqrt{2}/2); (i = 1, N) \quad (19)$$

The Mean Square Error (*MSE*) is calculated between the criteria values $f_i^0 (i = 1, N)$ for the Pareto – compromise design and the corresponding criteria values $f_i^* (i = 1, N)$ for each of the *m* original Pareto designs as follows (Grierson, 2008):

$$MSE = 1/N \sum \left(1 - f_i^* / f_i^0\right)^2 ; (i = 1, N) \quad (20)$$

The smallest *MS*E value is considered as the best alternative design to the Pareto compromise design. A code named MO-OPTIM has been written for applying the previous mathematical formulation. Figure (1) shows the general flow chart for the MO-OPTIM code.

4 APPLICATION ON DAMNHOUR CITY NETWORK

Water distribution network of Damnhour district, Egypt, is divided into Damnhour city network and the rural network. This study depends only on the water distribution network for Damnhour city. There are eight pipelines connect the city network with the rural network. Therefore the city network separation is performed at these eight pipelines. Field measurement Locations are carried out at these eight pipelines. The available field measurements at these locations are flow rates and pressure heads. In the mathematical model of Damnhour city network, the eight separation points are considered as fixed grade points (boundary conditions) of known outflows from the city network to the rural network. Figure (2) shows Damnhour city network which contains 256 pipes and 193 nodes after performing the necessary simplifications in pipes and nodes. All data and field measurements are given by Damnhour master plan (2006).

There are two water treatment stations supplying the network with water and a portion of this water going to the rural network through eight pipelines connecting the city network with the rural network. These stations are the new Damnhour and Czech water treatment stations. It is worth mentioning that these two stations rely on the water from the east of El-Khandak canal, a branch of a Mahmoudia canal. Mahmoudia canal is considered as one of the distributaries of River Nile. The new Damnhour pump station consists of the following types of pumps: type (1) consists of 4 pumps with their rated operation point (400 lit/sec, 64 m), type (2) consists of 3 pumps with their rated operation point (300 lit/sec, 64 m), and type (3) consists of 2 pumps with their rated operation point (150 lit/sec, 64 m). The Czech pump station involves two types of pumps: type (4) consists of 2 pumps with their rated operation point (200 lit/sec, 60 m) and type (5) consists of 2 pumps with their rated operation point (100 lit/sec, 60 m), Damnhour master plan (2006).

It can be assumed that all nodes in Damnhour city network, other than connection nodes, have the same spatial pattern and then there is only one group for all demand nodes. Temporal Pattern for this one group of nodes can actually be computed as explained by El-Ghandour (2010).

The analysis is performed at the interval of the maximum difference between nodal pressure heads and minimum allowable pressure head (assumed 25.0 m). This interval is found to be from 3:00 a.m. to 6:00 a.m., according to the temporal pattern given by El-Ghandour (2010). The pump combinations from the five types in the two pump stations, to get the right scheduling horizon for this time interval, is found to be two pumps from type (1) with operation point (368.3 lit/sec, 67.3 m) and one pump from type (5) with operation point (93.9 lit/sec, 62.4 m), El-Ghandour (2010). This combination of pumps gives nearly the rated operation points, but the difference between nodal pressure heads and minimum allowable pressure head may cause an increase in leakage. Therefore, a method has to be existed to make the pumps operate in the pre-specified ranges to reduce the operation and maintenance costs, and in the same time, the minimum obtained nodal heads to be as near as possible to the minimum allowable head (25.0 m). The system of TCVs is suggested to solve this problem.

TCVs can be used to minimize excessive pressures in accordance with the changing demand pattern. These valves have variable degrees of closure that reduce the capacity of the pipe in which they are located and increase the pressure loss (Vairavamoorthy and Lumbers, 1998). The used TCV loss coefficient is proportional to the valve opening size according to the following equation (Awad, 2005):

$$f_{vij}(\theta) = \begin{cases} 165226 \times 10^{-0.18\theta} & (0 \le \theta_v < 13) \\ 3696 \times 10^{-0.06\theta} & (13 \le \theta_v < 40) \\ 221 \times 10^{-0.03\theta} & (40 \le \theta_v \le 100) \end{cases} \quad (21)$$

in which, f_{vij} = valve loss coefficient for the valve located at pipe between node i and node j, and θ_v = percentage of TCV opening size.

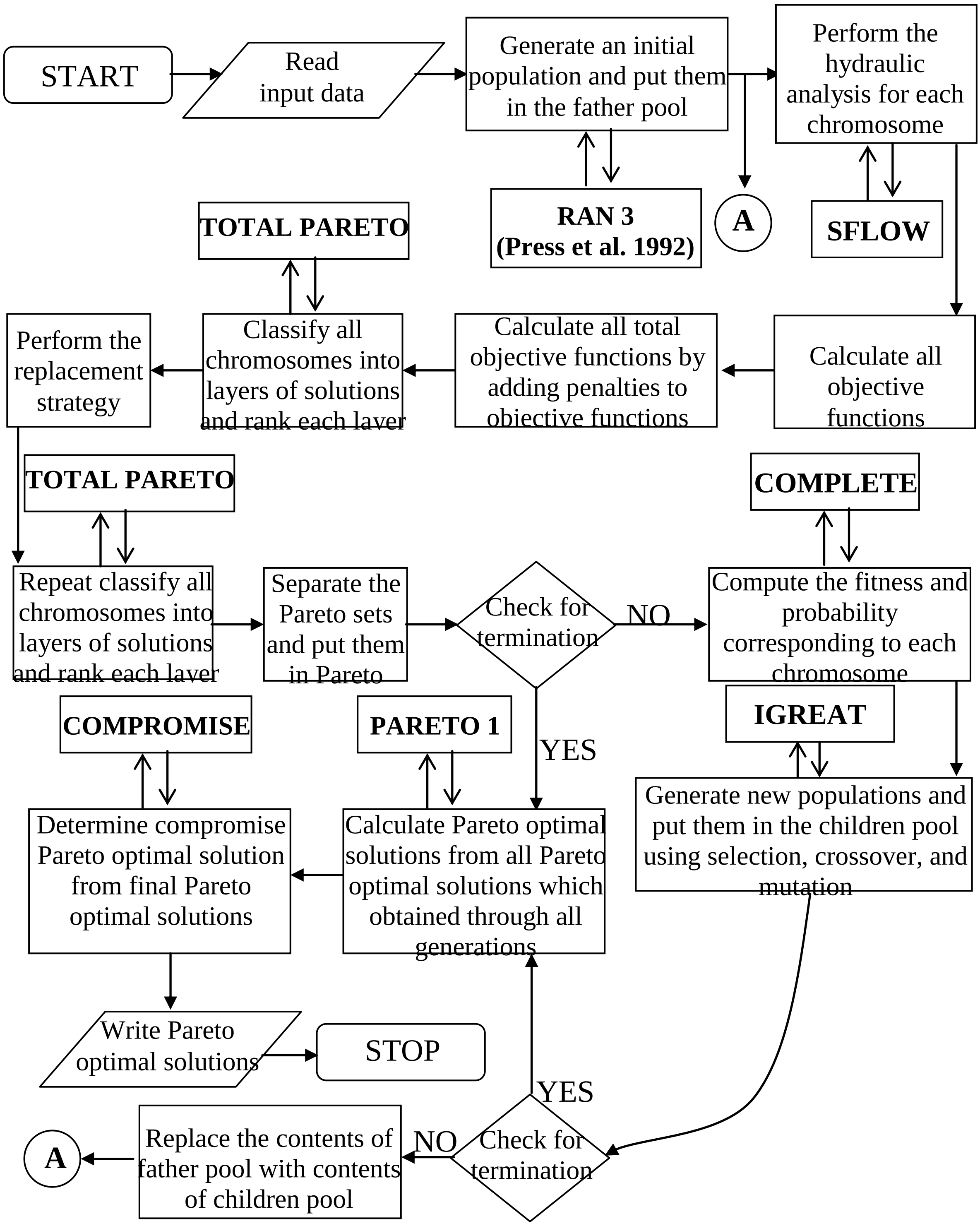

Figure 1. Flow chart for the MO-OPTIM code

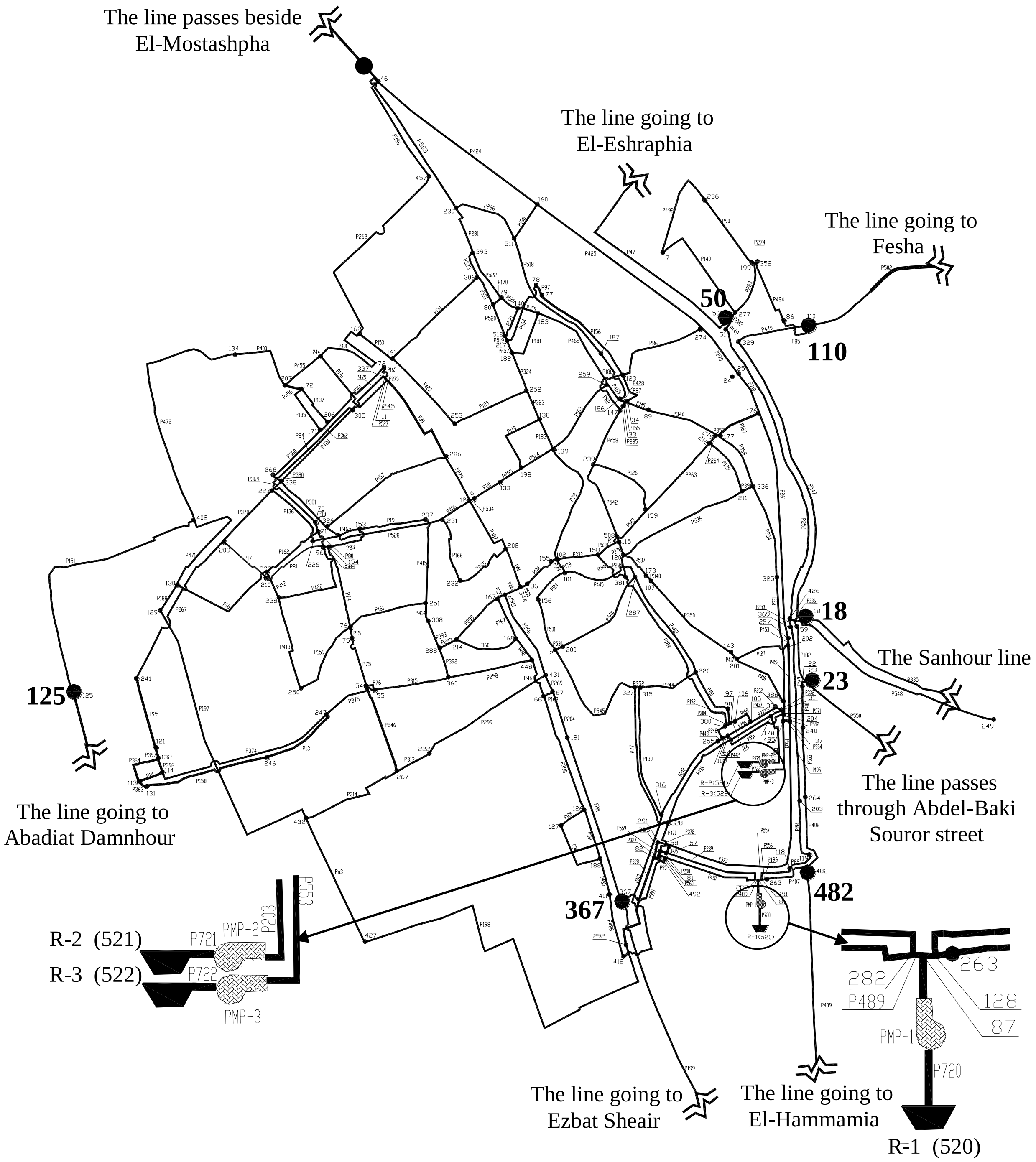

Figure 2. Layout of the Damnhour city water distribution network

MO-OPTIM code is used to determine best locations, minimum number of TCVs, and their opening size percentages by minimizing both valves number and the objective function, Eq. (1). The following limits are considered through the simulation: the minimum allowable nodal head is taken 25.0 m; the maximum allowable flow velocity is 2.0 m/s; and the maximum and minimum allowable heads are considered to be 70.0 m and 55.0 m for two pumps of the new Damnhour pump station while the corresponding heads for one pump of Czech pump station are 65.0 m and 50.0 m. The following genetic algorithm parameters are considered through the MO-OPTIM code: population size = 200; maximum number of generations = 500; crossover ratio = 0.7; mutation ratio = 0.08; the Idum number is taken (100); and uniform crossover is used.

Convergence index for any generation equals the minimum distance between the current Pareto optimal solutions and the corresponding ones at the previous generation dividing by the number of current Pareto optimal solutions (El-Ghandour, 2010). The convergence of the optimization process can be checked according to the change in a convergence index with the number of generation, as shown in Figure (3). The code terminates either the number of generation reaches to its maximum value, or the convergence index is less than or equal 0.0001 after repetition of this value for 20 times. According to this figure, it can be found that the convergence index is high and fluctuates before number of generation equal to 50 and then gradually decreases to reach number of generation equal to 150. After number of generation equal to 150, the convergence index is less than 0.1. Fluctuations in the figure may be due to mutation. However, the code reaches to the maximum number of generations (500 generations) without achieving termination criterion for convergence index. This is due to either the used small tolerance of convergence index (0.0001) or condition of repeating the convergence index is less than the permissible tolerance (20 times). Figure (4) shows all Pareto optimal solutions which are obtained at each number of 500 generations. From this figure, the random and scattered points indicate that Pareto optimal solutions at initial generations while the crowded points indicate the Pareto optimal solutions at final generations. All Pareto optimal solutions at each generation are competing in order to reach the final Pareto optimal solutions as shown in this figure.

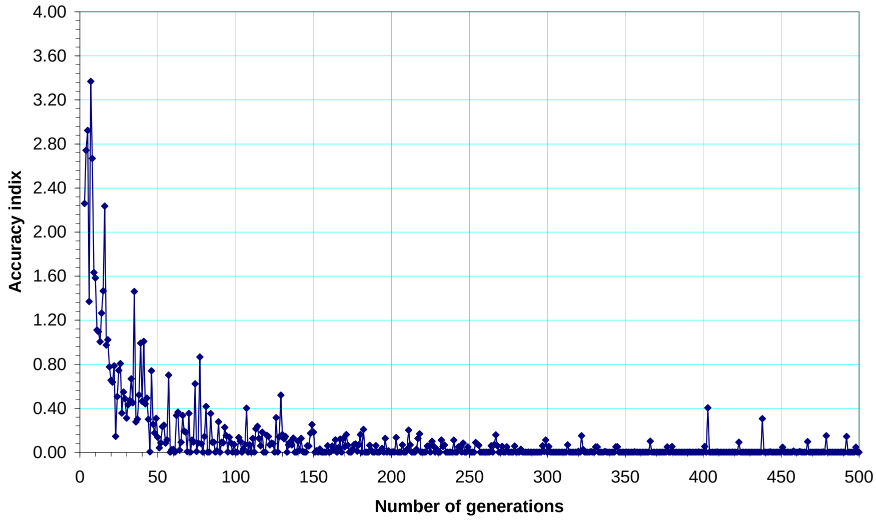

Figure 3. Relationship between accuracy index and number of generations

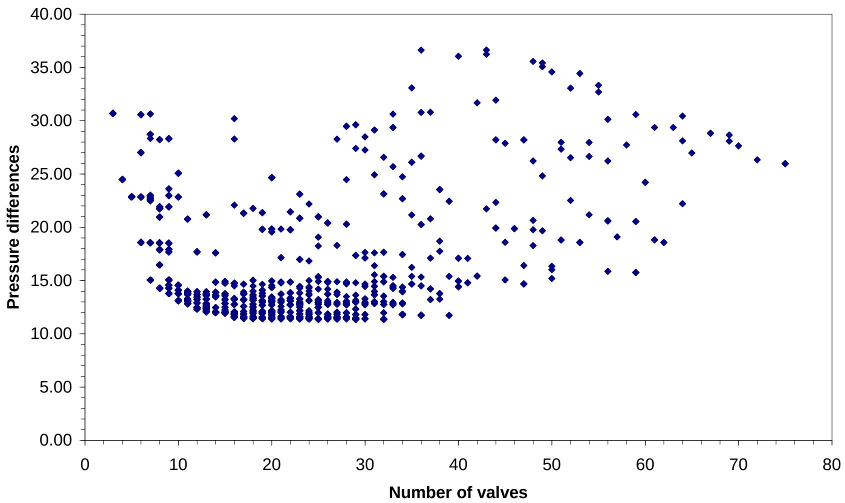

Figure 4. Total Pareto optimal solutions at all generations

Figure (5) presents results of the final Pareto optimal solutions. Each point in the figure represents a solution for the problem which contains number and suitable locations of valve combinations. From this figure it can be noticed that, after a number of valves equal to (10) the reduction of network pressures is rather insignificant and it can be concluded that reduction of excessive pressures can be obtained with the smallest number of valves when they are optimally located in the network which is the same conclusion presented by Reis at al. (1997).

Values of compromise solution among this set of Pareto – optimal solutions, shown in Figure (5), are found to be 14.51 m for pressure head differences and 7.7 for the number of valves. The existed Pareto optimal solutions in Figure (5) do not include a solution of these two values, and then a mean square error, Eq. (20) is used for all solutions to determine the best four solutions (according to the mathematical point of view) close to the compromise one. Table (1) presents the best four alternatives which have least mean square error and their values. Each alternative in this table represents a suitable solution which includes the number and location for a set of valves.

It is noticed that, the performance of the four alternatives are nearly the same and the solution which has 8 valves is found to be the best. Therefore this result confirms the previous conclusion. The locations of these eight valves are at pipes P201, P332, P452, P482, P489, P490, P557, and P558. These pipes represent the main pipes located near the two pump stations and any change in their head losses causes a significant reduction of network nodal heads. Table (2) exhibits the optimal opening size for every valve of the eight valves at the time interval under consideration (from 3.0 a.m. to 6.0 a.m.). From this table it can be seen that valve opening sizes are small. This is an expected behavior as increasing the excessive pressures at this interval as a result of decreasing the total network demands. Head and discharge of each operated pump through this interval are again computed and are found to be (354.2 lit/sec and 68.8 m) for the two operated pumps in the new Damnhour station and (122.8 lit/sec and 50.2 m) for the one operated pump in the Czech station. The deviations between the pumps operation points, in this case, and case of there, are no valves, due to the presence of valves at the main pipes which are located near the two pump stations. Existing valves on these pipes change their head losses and consequently operation point of each pump is changed.

To show the efficiency of this solution in reducing the excessive pressures, the simulated nodal head distribution without and with using TCVs system are shown in Figure (6). The system of TCVs enables the pumps to operate in the required range and in the same time, the excessive pressures are reduced.

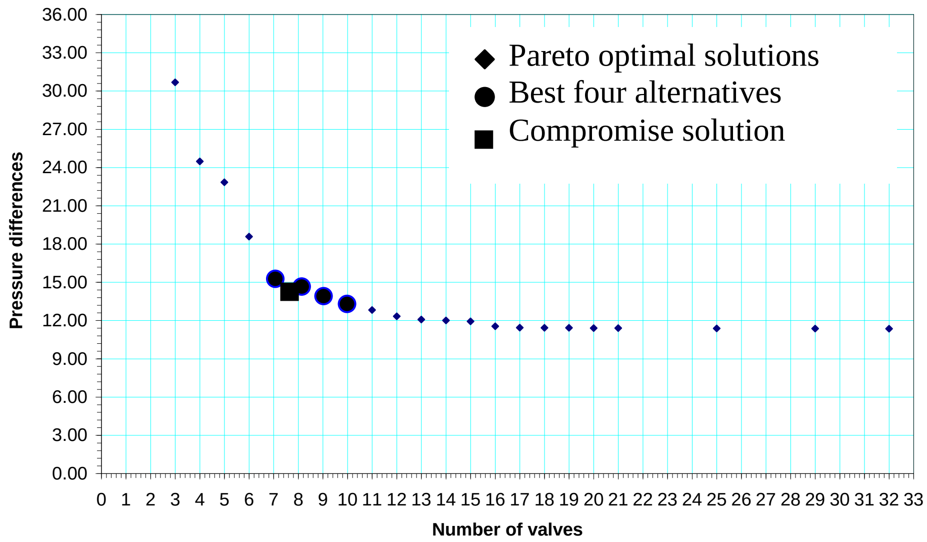

Figure 5. Final Pareto optimal solutions

Table 1. Data of the best four alternatives

Alternative	Criterion (1) Pressure head differences (m) Figure (8.4)	Criterion (2) Number of valves	MSE, Eq. (20)
1	15.04	7	0.005081
2	14.28	8	0.000760
3	13.79	9	0.014853
4	13.11	10	0.048015

Table 2. Optimal valve opening size percentage during the day

Pipe ID Field	Pipe ID model	Valve opening size percentage Time interval from3.00 a.m. to 6.00 a.m.
P201	78	9.6
P332	128	6.1
P452	188	0.7
P482	201	3.9
P489	206	16.2
P490	207	0.4
P557	243	9.2
P558	244	2.1

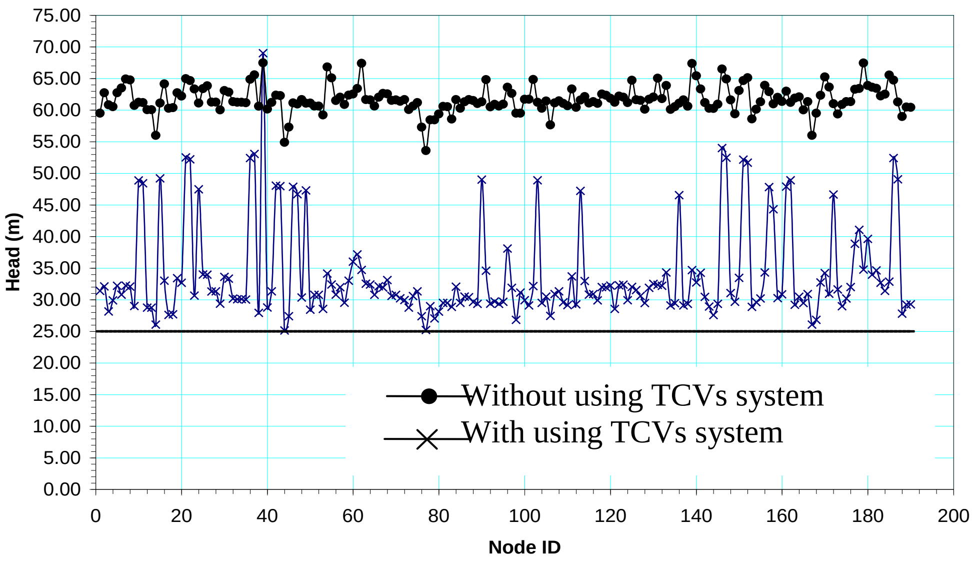

Figure 6. Simulated nodal head distribution for time interval between 3 a.m. and 6 a.m.

An estimation of the total amount of leakage is calculated for both the controlled and uncontrolled pressures. The following empirical equation, based on field data, may be used to determine the total leakage volume from a pipe connecting node *i* with node *j*. It can incorporate any openings or cracks in any pipe (Jowitt and Xu, 1990; Reis et al., 1997; Vairavamoorthy and Lumbers, 1998; and Awad, 2005).

$$LV_{ij} = K_{cij}\, L_{ij}\, P_{ij}^{1.18} \quad (22)$$

in which, LV_{ij} = total leakage volume through the pipe, K_{cij} = unknown experimental coefficient depends on the value of service pressure, age of the pipe, deterioration of the pipe and the soil properties, L_{ij} = length of the pipe, and P_{ij} = average service pressure of the pipe.

To overcome the difficulty of determining the coefficient K_{cij} , water leakage calculation in this study is computed in the term of it, by considering a constant value of this coefficient through all network. Leakage volumes are 8403253.8 K_c and 4884449.0 K_c for uncontrolled and controlled cases respectively. Reduction percentage of leakage is computed according the following equation and equal to 42.0 %.

$$\text{Leakage reduction precentage} = (LV_{un} - LV_{c\ i}) / LV_{un} \quad (23)$$

in which, $LV_{un\ i}$ = uncontrolled total leakage volume through the network at interval i, $LV_{c\ i}$ = controlled total leakage volume through the network at interval i, and N_{int} = number of time interval.

To obtain the maximum leakage reduction throughout the day, optimal opening size for each valve can be determined at each time interval during the day. This is performed by dividing the extended period problem into a series of optimization ones corresponding to each time interval using single objective genetic algorithm technique.

5 CONCLUSIONS AND RECOMMENDATIONS

Number and locations for the system of TCVs, as well as their opening size percentages, can be determined in one run to optimally reduce the excessive pressure heads, and consequently leakage reduction, under several constraints using MO-OPTIM code. This code has been established initially to apply the principles of the multi-objectives genetic algorithm and two new techniques. The first one is used for pipe network steady state analysis as given by El-Ghandour (2010), while the second technique is applied a Grierson (2008) methodology for obtaining a compromise solution from Pareto optimal solutions which obtained from multi-objective solution. The presented code enables the pumps to operate in pre-specified ranges close to the pump rated operation points. Application of this code on Damnhour city water distribution network shows that, reduction of excessive pressures can be obtained with the smallest number of valves when they are optimally located in the network. The previously mentioned results are very useful for the decision makers in El- Beheira Company for potable water and domestic sewage for optimal management of the Damnhour network pressure head and consequently leakage reduction.

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NOTATION AND ABBREVIATIONS

Notation:

The following symbols are used in this paper:

E_p : the energy supplied by a pump;
F_i : the fitness;
f_{vij} : valve loss coefficient for the valve located at pipe between node *i* and node *j*;
h_f : head loss due to friction in a pipe;
h_j : head at node *j*;
h_j^T : required target head at the same node;
H_{min} : minimum allowable head;
h_{pi} : head produced by a pump *i*;
$h_{Pi\,min}$: minimum allowable pump head for each pump *i*;
$h_{Pi\,max}$: maximum allowable pump head for each pump *i*;
K_{cij} : unknown experimental coefficient;
L_{ij} : length of the pipe;
$LV_{c\,i}$: controlled total leakage volume through the network at interval *i*;
$LV_{un\,i}$: uncontrolled total leakage volume through the network at interval *i*;
LV_{ij} : total leakage volume through the pipe;
NP : number pumps through the network;
n : number of nodes in the network;
N_{int} : number of time interval;
N_V : number of valves;
P : number of pipes through the network;
P_{ij} : average service pressure of the pipe;
Q_e : external inflow or demand at the junction node;
Q_{in} : flow into the junction;
Q_{out} : flow out of the junction;
$rank_i$: the rank number of individual *i*;
V_{max} : maximum allowable flow velocity;
V_k : velocity of flow through pipe *k*; and
θ_v : percentage of TCV opening size

Abbreviations:

The following abbreviations are used in this paper:

CVs : Check Valves;
ELGT : Extended Linear Graph Theory;
GA : Genetic Algorithm;
GAP : Genetic Algorithm Processor;
LTM : Linear Theory Method;

MCDM	: Multi-Criteria Decision Making;
RMSE	: Root Mean Square Error;
PRVs	: Pressure Reducing Valves;
PSVs	: Pressure Sustaining Valves;
SQP	: Sequence of Quadratic Programming;
TCVs	: Throttle Control Valves; and
WDS	: Water Distribution System.