MULTI-OBJECTIVE SOLUTION: PUMPS OPERATION COST AND LEAKAGE REDUCTION

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ABSTRACT

This research paper tries to treat the problem of water leakage, via pressure regulation, in conjunction with the problem of increased pumping costs at water distribution networks. The objective of this study is to minimize both the excessive nodal pressure heads and total pumps operation cost. 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, using system of Pressure Reducing Valves (PRVs), satisfying all hydraulic constraints. Multi-objective genetic algorithm technique is linked with both a new techniques, the first one used for simulation of pipe network in steady state conditions while the second technique used for determination of a compromise solution from a set of Pareto optimal solutions. The great advantage of the proposed methodology is simultaneously solving the above mentioned two problems instead of handling each problem separately as carried out by other researchers. At the same time, One Pareto optimal solution for the multi-objective problem is determined instead of several trade-off alternatives. A computer code, called MO-OPTIM, has been originally written to apply the mathematical principles of the problem. This code is 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.

1 INTRODUCTION

When handling the problem of optimal pump scheduling throughout the day to reduce the total pumping cost, especially in case of there are no elevated tanks in the pipe network, the difference between minimum deduced nodal heads values and the minimum allowable head significantly increases through night time intervals. This is behavior due to decrease of the total demands through these intervals and the nodal heads are restricted by the pump heads which gives nearly the rated operation points, but this difference may cause an increase of leakage problem. Therefore, a method have to be proposed to make the pumps operate in the pre-specified ranges in order 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. Tremendous research works have been treated each problem separately for the two problems (increasing pumping cost and leakage problems) as follows:

1.1. Optimal Scheduling of Pumps for Reduction of Pumping Cost

The energy required to operate pump stations can account for a significant amount of electrical consumption in a municipality. Attempts to improve pump operation efficiency focus on three different aspects: inefficient pump combinations, inefficient pump scheduling (which is the important element in this research), and inefficient pump performances (Brion and Mays 1991). Pump scheduling (e.g. operation policy for a pump station) represents a set of temporal rules or guidelines (individual pump operating times) that indicate when a particular pump or group of pumps should be turned on and off over the control period (typically 24 hr).

Brion and Mays (1991) used nonlinear programming methodology for optimal scheduling of pumps to achieve minimum operation cost under a given set of operating conditions. The proposed methodology was applied on a pressure zone of the Austin, USA. Jowitt and Germanopoulos (1992) adopted linear programming method for determining an optimal (minimum cost) schedule of pumping on a 24-hr basis. Both unit and maximum demand electricity charges were taken into consideration. Nitivattananon et al. (1996) presented a Dynamic Programming (DP) model to generate pump schedules in real-time operation for a complex water supply system. Long-and short-term models were included for obtaining monthly policy and daily pump schedules respectively. Boulos et al. (2001) implemented an optimal operation model, called H_2ONET scheduler, for real-time control of multi-source, multi-tank water distribution systems. The mathematical model used the latest advances in genetic algorithms for optimal operation of pump stations with the objective of minimization of the energy consumed for pumping. Moradi-Jalal et al. (2003, 2004) used both a nonlinear programming model and WAPIRRA Scheduler model which depends on a genetic algorithm for the optimal design and operation of pumping stations. The objectives of the two studies were to select pump type, capacity, and number of units as well as scheduling the operation of irrigation pumps for the purpose of the minimum design and operation costs for a given set of demand curves. Lucken et al. (2005) presented a model which uses parallel asynchronous evolutionary algorithms as a tool to aid in solving an optimal pump scheduling problem. The objectives of the study were to minimize the followings: electric energy cost, maintenance cost, maximum power peak, and level variation in a reservoir. Ibanez et al. (2005) linked a multi-objective model called SPEA2 with the hydraulic simulator EPANET to minimize both the electricity cost and number of pump switches. A feasibility handling technique designed for multi-objective optimization and based on the dominance criteria was used to replace the penalty function and reparation mechanisms. Shu et al. (2007) exhibited a methodology based on hybrid genetic algorithm, which combines genetic algorithm and simulated annealing algorithm, for optimal control large-scale water distribution network. The objective of the study was to minimize the total operation cost of the pumps satisfying all hydraulic constraints. Bounds et al. (2007) adopted nonlinear programming model to minimize both pumping cost and source cost while satisfying pressure limits. The proposed method took into account the pump head-flow characteristics, electrical tariffs, and reservoir levels. Sousa et al. (2007) demonstrated a multi-objectives methodology to optimize both pumping costs and hydraulic reliability of the system in terms of available water volume in downstream storage tanks. Pasha and Lansey (2009) used linear programming (LP) method to formulate and solve single tank system for the optimal pump scheduling with the objective of minimizing energy cost. The pump stations relationships were linearized using relationships among energy required, pumping flow, demand factors, and tank storage or tank water levels.

1.2. Leakage Management

Leakage in water supply networks represents a large percentage of the total water supplied, 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 water supplied and the amount of water consumed. Leakage is one of the components of water loss and comprises physical losses from pipes, joints and fittings and also over flows 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 system of 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 tend 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 can 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 was considered as an approximation for the original problem, to generate at each step a search direction that is used to update the solution vector. 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 greater 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. Multi-objective genetic algorithm was used to determine 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.

The main purpose of this research paper is to suggest a methodology using multi-objective genetic algorithm optimization technique for determination of both scheduling of pumps and setting heads for pre-specified number of PRVs in water distribution systems. The objective of the optimization problem is to minimize both increasing in pumping cost and undesirable excessive nodal pressure heads (and consequently leakage reduction).

2 MATHEMATICAL FORMULATION

This section presents a mathematical formulation for the problem under consideration which consists of objective functions, constraints, and decision variables.

2.1. Objective Functions

The first objective function, concerns with optimal scheduling of pumps, is the minimization of the consumed energy cost. Mathematically, it can be expressed as follows (Brion and Mays 1991):

$$Obj_1 = \sum_{t=1}^{T} \sum_{p=1}^{NP} \frac{\gamma q_p h_p}{\eta_{pt}} * D_{pt} * UC_t \tag{1}$$

in which, γ = specific weight of water (*9.81 KN/m³*), q_p= discharge of pump during time period t (m^3/s), h_p= head of pump during time period t (m), η_{pt} = efficiency of pump p in the time period t, D_{pt} = length of time for pump p operates during time period t (hr), UC_t = unit pumping cost (L.E./kw.hr) during time period t, T= number of time intervals through the day, and NP= number of working pumps.

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

$$Obj_2 = \sum_{i=1}^{T} \left(\left[\frac{1}{n} \sum_{j=1}^{n} \left(h_j - h_j^T \right)^2 \right]^{1/2} \right) \tag{2}$$

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

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:

$$h_j(t) \geq H_{min}, \qquad j = 1, \ldots\ldots\ldots n, \; t=1,\ldots\ldots.T \tag{3}$$

$$V_k(t) \leq V_{max}, \qquad k = 1, \ldots\ldots\ldots p, \; t=1,\ldots\ldots.T \tag{4}$$

$$h_{Pi\,min} \leq h_{pi}(t) \leq h_{Pi\,max}, \qquad i = 1, \ldots\ldots\ldots NP, \; t=1,\ldots\ldots.T \tag{5}$$

in which, h_j = pressure head at node j; H_{min} = minimum allowable head*;* n = number of nodes through the network; T = number of time intervals through the day; 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 divided into two categories as follows: The first one are used to specify the pump control setting values for each time interval. The control setting value means that, the situation of the pump on or off.

$$S_k(t) = \{0,1\} \qquad k = 1, \ldots\ldots\ldots NP, \; t=1,\ldots\ldots.T \tag{6}$$

in which, $S_k(t)$ = control setting of pump k at the time interval t and takes a value of either 0 (pump off) or 1 (pump on).

The second category can be used to set limits on PRVs setting heads as follows:

$$H_{set\,min} \leq H_{set\,v}(t) \leq H_{set\,max},, \qquad N_V = 1,\ldots\ldots\ldots V, \; t=1,\ldots\ldots.T \tag{7}$$

in which, $H_{set\,v}(t)$ = head at downstream valve v at the time interval t; $H_{set\,min}$ and $H_{set\,max}$,= minimum and maximum allowable heads at downstream of each PRV; and N_V = number of PRVs.

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:

$$\sum Q_{in} - \sum Q_{out} = Q_e \tag{8}$$

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.

$$\sum h_f = zero \quad \textit{(around each loop in case of there is no pump)} \tag{9}$$

$$\sum h_f = E_p \quad \textit{(if there is a pump)} \tag{10}$$

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 can be divided into two types as follows:

The first one related to pumps, in which the decision variables/unknowns are the switchings of each pump. Then, for each pump during a certain time interval, the operation policy can be represented by (0) if the pump is off during that time interval whereas is (1) in case of pump operation. The number of decision variables is equal to (*NP*T*) where, *NP* means number of pumps in the network and *T* is the number of time intervals through the day. In the proposed code, the day is divided into equally time intervals therefore; the length of time period for each working pump is obtained by adding the working intervals, or in other words the sum of intervals that have number (1) is used for the pump under consideration.

The second type concerns with PRVs, in which Decision variables/unknowns are number of valves as well as their locations and setting of each corresponding to each time interval.

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 get 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 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. 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 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 number of genes equal to the number of unknown variables.
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):

   $$F_i = 1/rank_i \qquad (11)$$

   in which, F_i and $rank_i$ = the fitness and the rank number of individual i.
7. *Aggregation of Pareto solution:* for each generation, a set of Pareto solutions that has 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. (11), using the roulette wheel method (Goldberg 1989).
   - Crossover: generates 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 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, mutation has to be applied on this gene. This mutation operates randomly by a new value for this gene.

The previous operators have to be repeated iteratively for each new 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 are reached to the maximum generation number or 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*".

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 $\{f_1(z), \ldots\ldots\ldots, f_n(z)\}$ subjected to $z \in \Omega$ (12)

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 of the Pareto design optimization problem, Eq. (12), represented by the set of m – dimensional objective criteria vectors $f_i^*(i = 1, \ldots.., N)$ 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:

$$X_i = \frac{\left(f_i^* - f_i^{\min}\right)}{\left(f_i^{\max} - f_i^{\min}\right)};\left(i = 1,\ldots.,N\right) \quad (13)$$

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 vector x^*, y^*.

$$x^* = \left(x + \delta x\right) / \left(1 + \delta x\right) = \left[x^{*\min},\ldots.., x^{*\max}\right]^T = \left[1 - \sqrt{2}/2,\ldots.,1\right]^T \quad (14.a)$$

$$y^* = \left(y + \delta y\right) / \left(1 + \delta y\right) = \left[y^{*\max},\ldots.., y^{*\min}\right]^T = \left[1,\ldots...,1 - \sqrt{2}/2\right]^T \quad (14.b)$$

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

$$\Delta x_0 = \Delta y_0 = 0.5 - \left(x_j^* - x_{j+1}^*\right)\left(y_j^* - y_{j+1}^*\right) / \left(x_j^* + x_{j+1}^* + y_j^* + y_{j+1}^*\right) \quad (15)$$

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

$$\Delta r_0 = \sqrt{2\Delta x_0} = \sqrt{2\Delta y_0} \quad (16)$$

The previous equations explain how to add several effects to the Pareto optimal solutions after normalization to take the shape of quarter of circle. The compromise solution exists on the quarter of 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 on 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:

$$x_i = \left[X_i^{\min},\ldots...,X_i^{\max}\right]^T = \left[0,\ldots..,1\right]^T;\left(i = 1,2,\ldots N\right) \quad (17)$$

$$Y_i = \left(\sum_{k=1}^{n} x_k - x_i\right) / \left(n - 1\right);\left(i = 1,2,\ldots..,N\right) \quad (18)$$

$$y_i = \left[X_i^{\max},\ldots...,X_i^{\min}\right]^T = \left[1,\ldots..,0\right]^T;\left(i = 1,2,\ldots N\right) \quad (19)$$

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:

$$f_i^0 = f_i^{\max} - \left(f_i^{\max} - f_i^{\min}\right)\left(\Delta r_i + \sqrt{2}/2\right);\left(i = 1, N\right) \quad (20)$$

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):

$$MSE = 1/N \sum \left(1 - f_i^* / f_i^0\right)^2 ; (i = 1, N) \tag{21}$$

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 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 pipe lines. Field measurement Locations are carried out at these eight pipe lines. 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 the 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 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).

There is one pipe line, with diameter equal to 1300 mm, outside from the new Damnhour pump station, while there are two pipe lines, with diameter equal to 600 mm for each, outside from the Czech pump station, Figure (2). Accordingly, the suitable pipes for containing PRVs are these three main ones outside from the existed two pump stations, and consequently the problem of determination of both optimal locations and number of valves is directly solved. In this case, it is suggested to use PRVs because the flow direction at these three pipes is known. The problem under consideration can be solved as multiple objectives one to determine both the best combination of operated pumps and optimal valve settings at each time interval, for reducing both the total operation cost and excessive pressures satisfying all hydraulic constraints.

Solving this problem as an extended period simulation in conjunction with multiple objectives faces a great difficulty to get an optimal solution, in order to tackle this problem both number of chromosomes and generations have to be increased which lead to a significant increase in computer running time. Alternatively, the problem of extended period can be divided into series of multiple objective optimization problems for each time interval. A pressure reducing valve (PRV) is designed to maintain a constant pressure at its downstream side, and is used in a situation where its downstream pressure is very high and there is no need for this pressure. In this case, PRV acts at each of the three main pipes as an ***organizer of pressure*** since the valve upstream pressure head enables each pump to operate in the required limits and on the other hand its downstream pressure head is sufficient to prevent any excessive pressures.

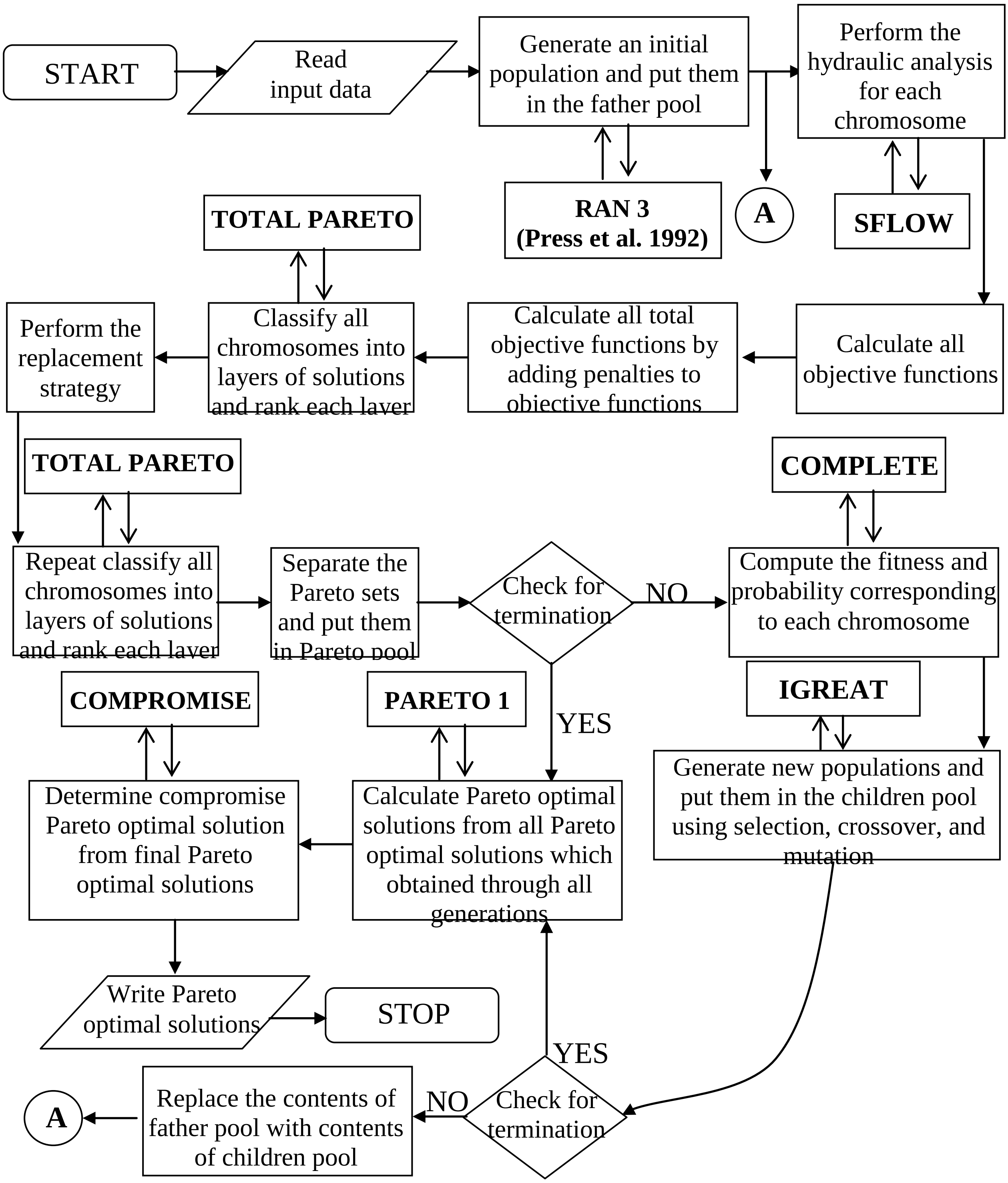

Figure (1): Flow chart for the MO-OPTIM code

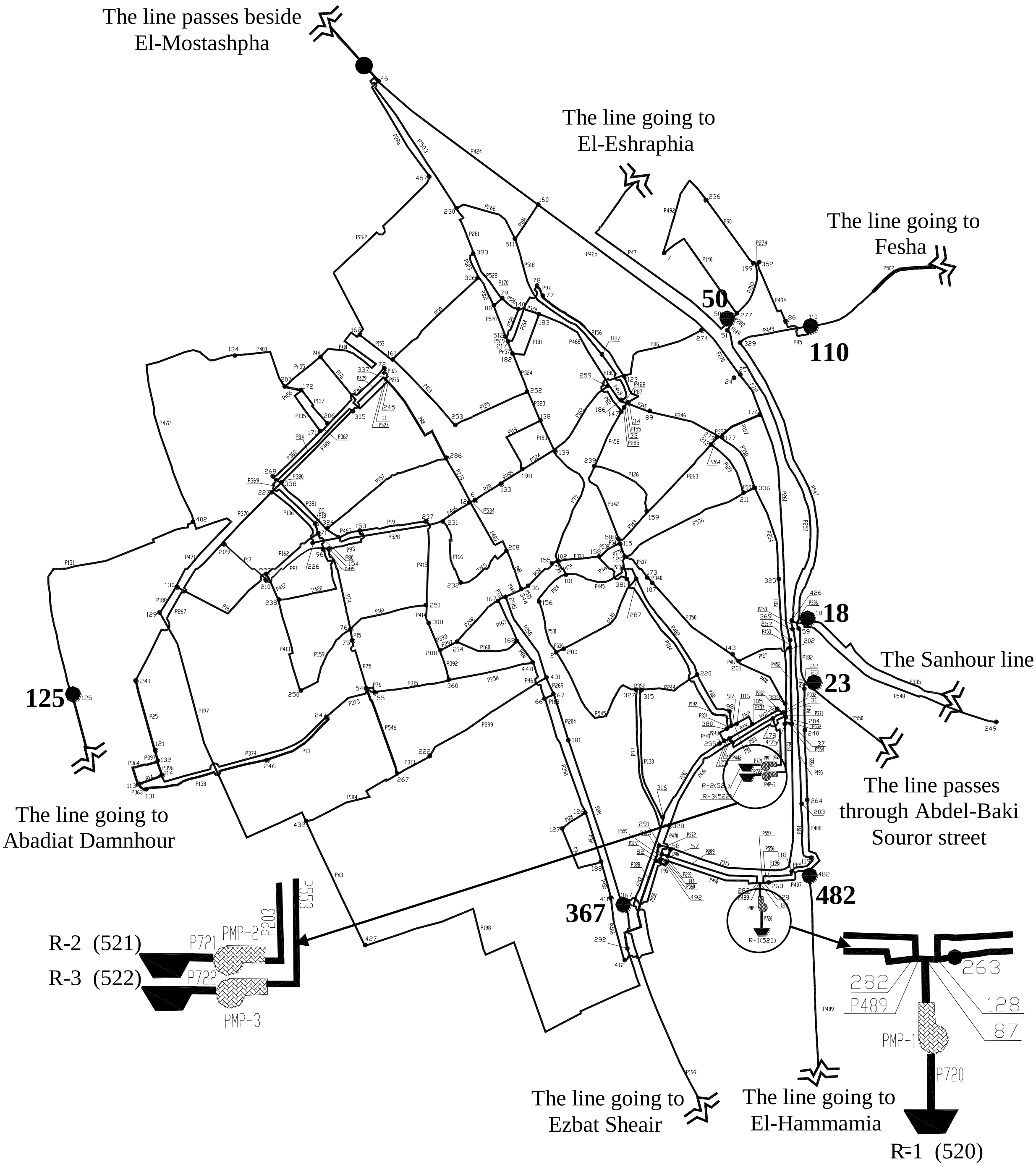

Figure (2): Layout of the Damnhour city water distribution network

The total simulation time of 24 hr is divided into eight 3-hr time intervals. The unit of pumping cost is constant during the day and is taken equal to (0.25 L.E./Kw hr)For every time intervals during the day, MO-OPTIM code is used to determine both right combinations of operated pumps, from the existed 13 pumps at the two pump stations, and suitable setting heads for the three PRVs by minimizing both Eqs. (1) and (2) satisfying constraints in Eqs. (3), (4), and (5).

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 dynamic heads for pumps of the new Damnhour pump station are 70.0 m and 55.0 m while the corresponding ones for all pumps of Czech pump station are 65.0 m and 50.0 m. The following parameters are considered through the MO-OPTIM code: population size = 100; maximum number of generations = 300; crossover ratio = 0.6; mutation ratio = 0.05; the Idum number is taken (100); and uniform crossover is used.

Figure (3) presents results of the final Pareto optimal solutions corresponding to every time interval. Each point in the figure represents a possible scenario for the problem, at the specified interval, which contains both suitable combination of operated pumps and setting heads for the three valves. The values of the compromise solutions, which have least mean square error, corresponding to every time interval are presented in Table (1).

Total pumping cost in this case is found to be 6036.8 L.E. /day or equal to 2,203,429.2 L.E./year. Extended period simulation results of the optimized Damnhour city network operation are presented in Table (2) which contains both a schedule for each pump and setting heads for each valve. From these results the followings are remarked:

- The shaded blocks represent switch on of the pump status while other blocks represent switch off of the pump.
- Pumping head across each pump through the day is found in the following ranges: (58.8 – 61.8) for pump (2), (60.7) for pump (3), (60.7) for pump (4), (55.9 – 63.9) for pump (5), (55.9 – 61.7) for pump (6), (55.9 – 61.7) for pump (7), (55.8 – 61.7) for pump (8), (55.8 – 63.8) for pump (9), (52.3 – 58.3) for pump (10), (52.3 – 58.3) for pump (11), (54.1 – 60.3) for pump (12), and (55.3 – 60.3) for pump (13).
- Maximum difference between upper and lower limits of operation heads, according to the previous remark is 8.0 m whereas the corresponding maximum difference in the previous solution is 11.0 m.
- It is possible to consider fixed operation point by taking the average value of the previous ranges for each pump but this slightly affect the minimum allowable head constraint. This step could be necessary for the application in the field.
- For new Damnhour pump station, Pump number (1) is considered as a standby pump, whereas it is required to construct two standby pumps, the first one with similar capacity for pumps number (5, 6, and 7) and the second pump with similar capacity for pumps numbers (8 and 9.)
- Czech pump station operates at full capacity (with all 4pumps) during the day; therefore two pumps of similar capacities have to be used as standing by units.
- Despite the number of pumps switching constraint is not taken into consideration but there are at least two rest periods for each pump.
- Values of valves setting heads at night intervals are less than the corresponding ones at day intervals due to increasing the excessive pressures at night intervals.
- Valve number (1) does not operate at time periods between12.0 p.m. and 3.0 p.m. and between 6.0 a.m. and 9.0 a.m.; valve number (2) does not operate at time period between 9.0 a.m. and 12.0 p.m.; and valve number (3) does not operate at time period between 6.0 a.m. and 9.0 a.m.
- For each PRV, the average value of setting heads can be computed for its similar setting heads values to simplify the field operation.

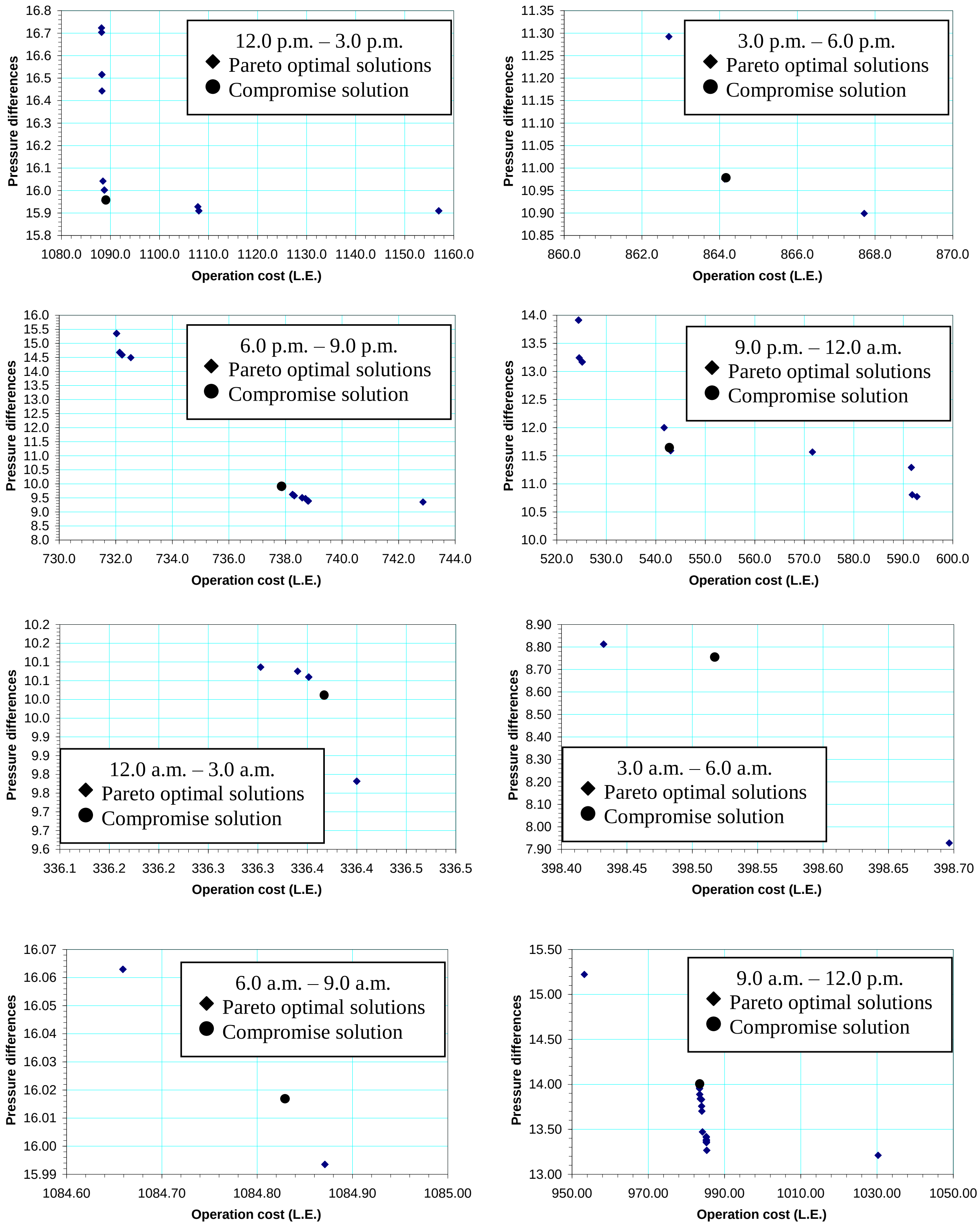

Figure (3): Final Pareto optimal solutions for each time interval

Table 1. Criteria values of each compromise solution for each time interval

Time interval	*Criterion 1 Pressure difference (m)*	*Criterion (2) Operation cost (L.E.)*	*MSE, Eq.(4.48)*
12.00 p.m. – 3.00 p.m.	15.95	1088.9	0.0000100
3.00 p.m. – 6.00 p.m.	10.98	864.1	0.0000080
6.00 p.m. – 9.00 p.m.	9.86	737.9	0.0100000
9.00 p.m. – 12.00 a.m.	11.62	542.8	0.0000003
12.00 a.m. – 3.00 a.m.	10.01	336.4	0.0000200
3.00 a.m. – 6.00 a.m.	8.76	398.5	0.0008000
6.00 a.m. – 9.00 a.m.	16.0	1084.8	0.0000003
9.00 a.m. – 12.00 p.m.	14.0	983.4	0.0000700

Table 2Results of pumps scheduling and pressure settings for three PRVs during the day

Pump type	***Pump No.***	***Interval (1) 12 p.m. – 3 p.m. Multiplier =2.2***		***Interval (2) 3 p.m. – 6 p.m. Multiplier =1.8***		***Interval (3) 6 p.m. – 9 p.m. Multiplier =1.5***		***Interval (4) 9 p.m. – 12 a.m. Multiplier =1.0***	
		Hset 1= -------- Hset 2= 52.8 m Hset 3= 50.9 m		Hset 1= 53.3 m Hset 2= 43.7 m Hset 3= 39.0 m		Hset 1= 47.6 m Hset 2= 41.4 m Hset 3= 37.8 m		Hset 1= 40.6 m Hset 2= 46.2 m Hset 3= 40.7 m	
type (1)	**Pump (1)**	Off	--------	Off	--------	Off	--------	Off	--------
			--------		--------		--------		--------
	Pump (2)	On	61.8 m	Off	--------	Off	--------	Off	--------
			419.7 l/s		--------		--------		--------
	Pump (3)	Off	--------	Off	--------	Off	--------	Off	--------
			--------		--------		--------		--------
	Pump (4)	Off	--------	Off	--------	Off	--------	Off	--------
			--------		--------		--------		--------
type (2)	**Pump (5)**	On	61.7 m	On	55.9 m	On	56.3 m	On	58.3 m
			315.1 l/s		354.0 l/s		351.0 l/s		338.1 l/s
	Pump (6)	On	61.7 m	On	55.9 m	On	56.3 m	On	58.3 m
			315.1 l/s		354.0 l/s		351.0 l/s		338.1 l/s
	Pump (7)	On	61.7 m	On	55.9 m	On	56.3 m	Off	--------
			315.1 l/s		354.0 l/s		351.0 l/s		--------
type (3)	**Pump (8)**	On	61.7 m	On	55.8 m	Off	--------	Off	--------
			158.1 l/s		177.4 l/s		--------		--------
	Pump (9)	Off	--------	On	55.8 m	On	56.2 m	Off	--------
			--------		177.4 l/s		176.0 l/s		--------
type (4)	**Pump (10)**	On	58.3 m	Off	--------	On	54.2 m	On	52.3 m
			207.9 l/s		--------		227.6 l/s		236.4 l/s
	Pump	On	58.3 m	On	60.3 m	Off	--------	On	52.3 m

	(11)		207.9 l/s		198.1 l/s		--------		236.4 l/s
type (5)	**Pump (12)**	On	58.3 m	On	60.3 m	On	54.1 m	Off	--------
			104.0 l/s		99.2 l/s		113.7 l/s		--------
	Pump (13)	On	58.3 m	On	60.3 m	Off	--------	Off	--------
			104.0 l/s		99.2 l/s		--------		--------

Table 2 Results of pumps scheduling and pressure settings for three PRVs during the day (continued)

Pump type	***Pump No.***	***Interval (5)*** ***12 a.m. – 3 a.m.*** ***Multiplier =2.2***		***Interval (6)*** ***3 a.m. – 6 a.m.*** ***Multiplier =1.8***		***Interval (7)*** ***6 a.m. – 9 a.m.*** ***Multiplier =1.5***		***Interval (8)*** ***9 a.m. – 12 p.m.*** ***Multiplier =1.0***	
		Hset 1= 36.7 m **Hset 2= 37.2 m** **Hset 3= 36.2 m**		**Hset 1= 36.3 m** **Hset 2= 37.4 m** **Hset 3= 36.0 m**		**Hset 1= -------** **Hset 2= 50.3 m** **Hset 3= -------**		**Hset 1= 57.6 m** **Hset 2= -------** **Hset 3= 44.4 m**	
type (1)	**Pump (1)**	**Off**	**--------**	**Off**	**--------**	**Off**	**--------**	**Off**	**--------**
			--------		**--------**		**--------**		**--------**
	Pump (2)	**Off**	**--------**	**Off**	**--------**	**On**	**60.7 m**	**On**	**58.8 m**
			--------		**--------**		**428.9 l/s**		**445.2 l/s**
	Pump (3)	**Off**	**--------**	**Off**	**--------**	**On**	**60.7 m**	**Off**	**--------**
			--------		**--------**		**428.9 l/s**		**--------**
	Pump (4)	**Off**	**--------**	**Off**	**--------**	**On**	**60.7 m**	**Off**	**--------**
			--------		**--------**		**428.9 l/s**		**--------**
type (2)	**Pump (5)**	**On**	**63.9 m**	**Off**	**--------**	**Off**	**--------**	**On**	**58.8 m**
			300.0 l/s		**--------**		**--------**		**334.3 l/s**
	Pump (6)	**Off**	**--------**	**On**	**57.7 m**	**Off**	**--------**	**On**	**58.8 m**
			--------		**341.6 l/s**		**--------**		**334.3 l/s**
	Pump (7)	**Off**	**--------**	**Off**	**--------**	**Off**	**--------**	**On**	**58.8 m**
			--------		**--------**		**--------**		**334.3 l/s**
type (3)	**Pump (8)**	**Off**	**--------**	**On**	**57.7 m**	**On**	**60.6 m**	**Off**	**--------**
			--------		**171.3 l/s**		**161.6 l/s**		**--------**
	Pump (9)	**On**	**63.8 m**	**Off**	**--------**	**Off**	**--------**	**Off**	**--------**
			150.6 l/s		**--------**		**--------**		**--------**
type (4)	**Pump (10)**	**Off**	**--------**	**Off**	**--------**	**On**	**55.6 m**	**On**	**55.6 m**
			--------		**--------**		**220.8 l/s**		**220.7 l/s**
	Pump (11)	**Off**	**--------**	**On**	**57.4 m**	**On**	**55.6 m**	**On**	**55.6 m**
			--------		**212.1 l/s**		**220.8 l/s**		**220.7 l/s**
type (5)	**Pump (12)**	**On**	**55.3 m**	**Off**	**--------**	**On**	**55.6 m**	**On**	**55.6 m**
			111.1 l/s		**--------**		**110.4 l/s**		**110.3 l/s**
	Pump (13)	**On**	**55.3 m**	**On**	**57.4 m**	**On**	**55.6 m**	**Off**	**--------**
			111.1 l/s		**106.1 l/s**		**110.4 l/s**		**--------**

To show the efficiency of this solution for reducing the excessive pressures, the simulated nodal head distributions are shown in Figures (4) to (11) for the two cases at each interval, the first one corresponds to the presented scenario without PRVs as given by EL-Ghandour (2010) and the second case is the current scenario.

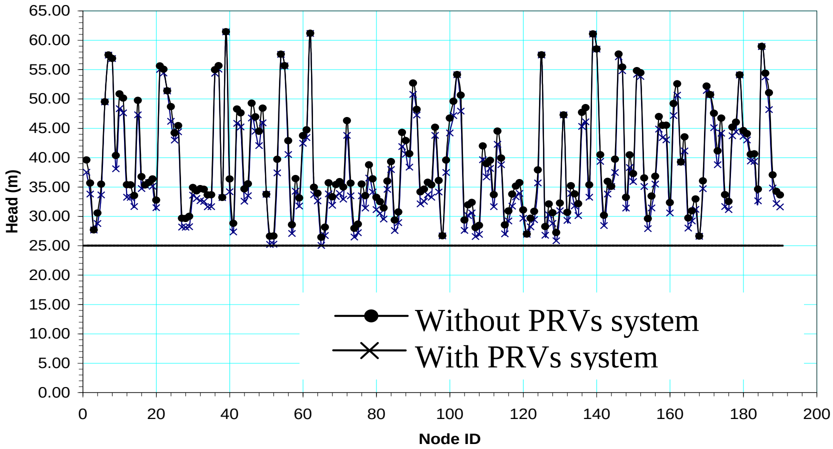

Figure (4): Simulated nodal head distribution for interval number (1) (12 p.m. – 3 p.m.)

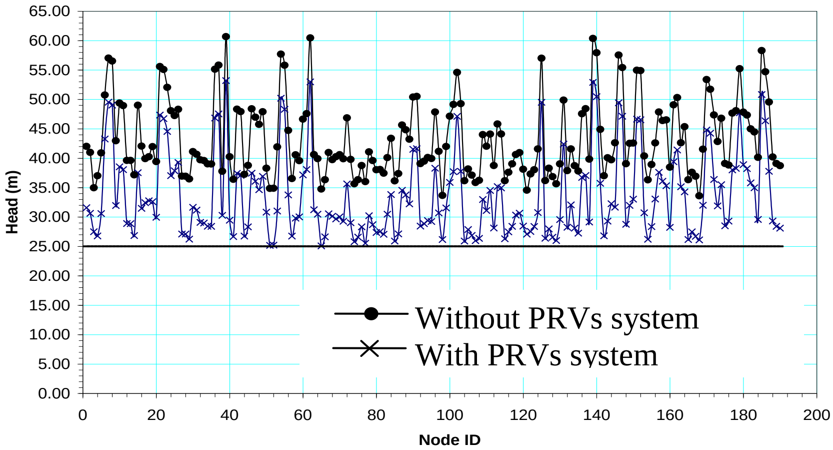

Figure (5): Simulated nodal head distribution for interval number (2) (3 p.m. – 6 p.m.)

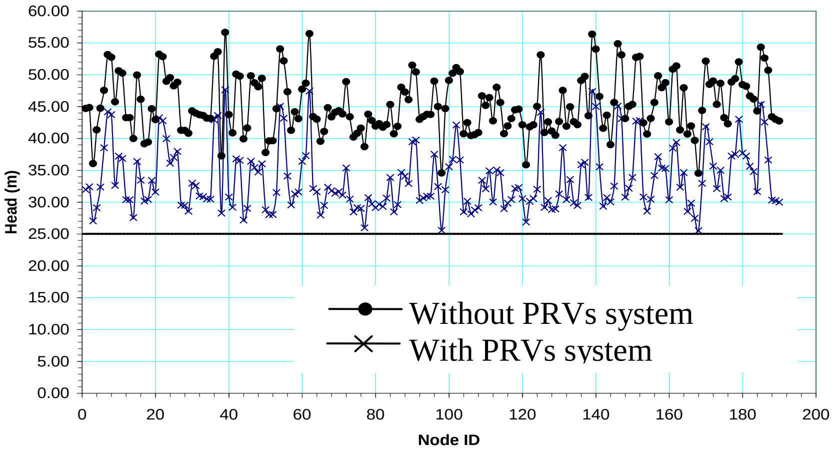

Figure (6): Simulated nodal head distribution for interval number (3) (6 p.m. – 9 p.m.)

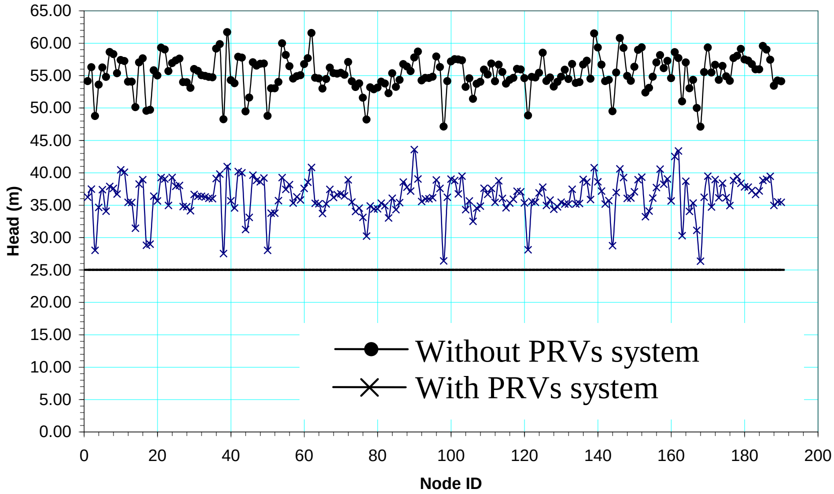

Figure (7): Simulated nodal head distribution for interval number (4) (9 p.m. – 12 a.m.)

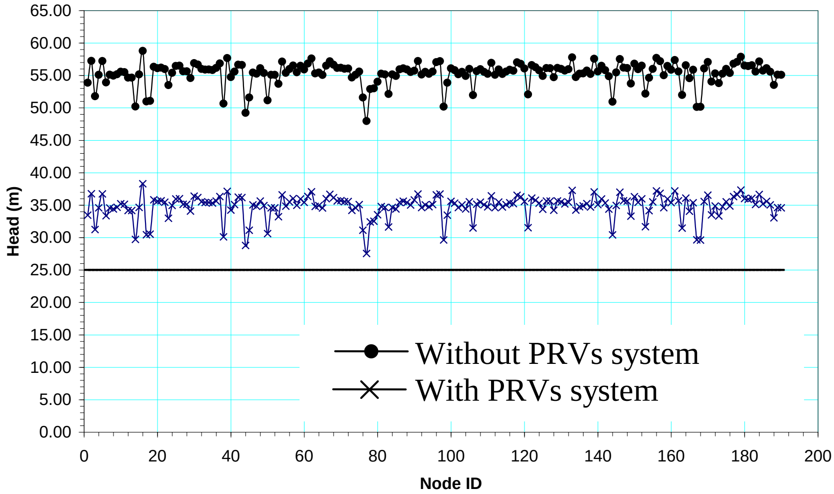

Figure (8): Simulated nodal head distribution for interval number (5) (12 a.m. – 3 a.m.)

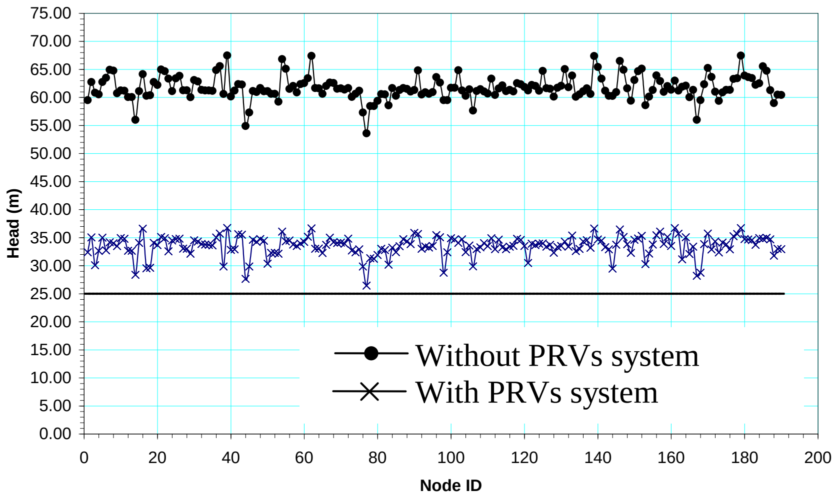

Figure (9): Simulated nodal head distribution for interval number (6) (3 a.m. – 6 a.m.)

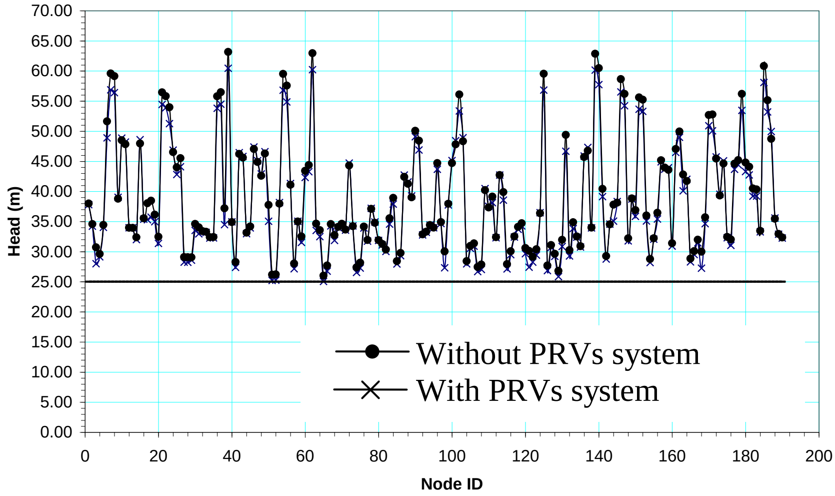

Figure (10): Simulated nodal head distribution for interval number (7) (6 a.m. – 9 a.m.)

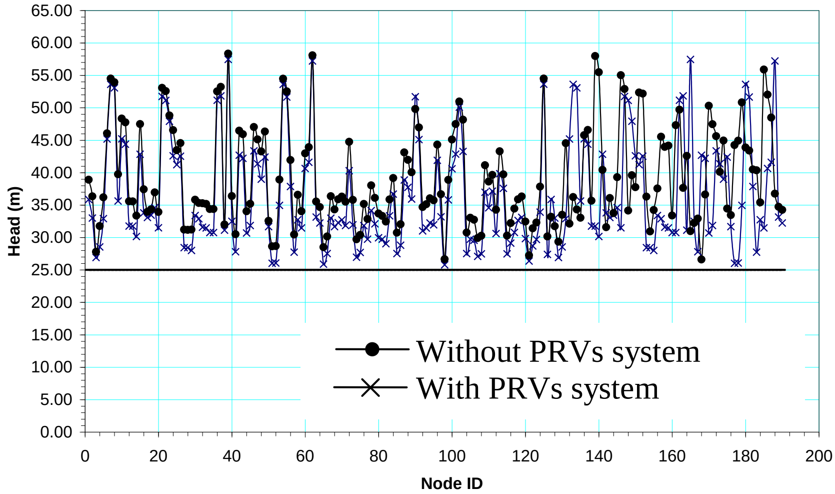

Figure (11): Simulated nodal head distribution for interval number (8) (9 a.m. – 12 p.m.)

An estimation of the total amount of leakage is calculated for both the two cases of 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).

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

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 as a ratio to the average leakage volume for the controlled case by considering a constant value of this coefficient through all network. Figure (12) shows the reduction of leakage obtained by introducing PRV to the network. The followings exhibit the used steps for drawing this figure.

- Computing the total leakage volume as a function of K_{cij}, Eq. (22), corresponding to the uncontrolled (without valves) and controlled case for each time interval of three hours (8 time intervals).
- Computing the average value for the leakage volume computed in the foregoing step for both the controlled case for each time interval.
- Dividing the value of leakage volume for each time interval for the controlled and uncontrolled cases, computed in the first step, by the corresponding average value of controlled cases, computed in the second step.

Reduction percentage of leakage is computed and listed in Table (3) for each time interval according the following equation:

$$Leakage\ reduction\ precentage = (LV_{un\ i} - LV_{c\ i})/\ LV_{un\ i} \qquad i=1,\ldots\ldots, N_{int} \qquad (23)$$

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.

It can be noticed from Table (3), maximum reduction percentage occurs at time interval between 3.00 a.m. and 6.00 a.m. while the minimum reduction percentage occurs at peak demand at time intervals between 12.00 p.m. and 3.00 p.m. and between 6.00 a.m. and 9.00 a.m.

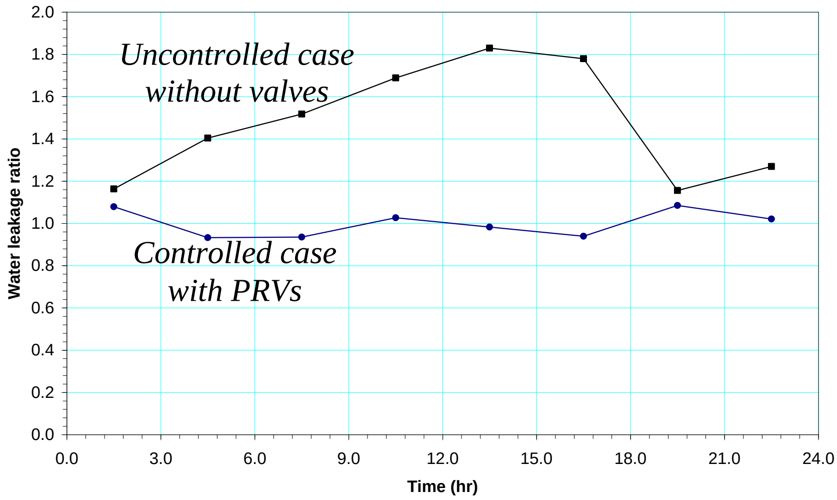

Figure (12): Reduction of leakage using PRVs system

Table (3): Reduction percentage using PRVs at each time interval

Time interval	*Reduction percentages of leakage using (PRVs)*
12.00 p.m. – 3.00 p.m.	7.2
3.00 p.m. – 6.00 p.m.	33.6
6.00 p.m. – 9.00 p.m.	38.3
9.00 p.m. – 12.00 a.m.	39.2
12.00 a.m. – 3.00 a.m.	46.3
3.00 a.m. – 6.00 a.m.	47.2
6.00 a.m. – 9.00 a.m.	6.2
9.00 a.m. – 12.00 p.m.	19.6

5 CONCLUSIONS

Optimal scheduling of pumps and setting heads for PRVs can be successfully determined to optimally reduce both the increasing pumping cost and excessive pressure heads for leakage reduction under several constraints using MO-OPTIM code. This code has been originally established 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, maximum leakage reduction percentage for Damnhour network may occur at time interval between 3.00 a.m. and 6.00 a.m. while the minimum reduction percentage may occur for the two techniques at peak demand at time intervals between12.00 p.m. and 3.00 p.m.; and between 6.00 a.m. and 9.00 a.m.

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