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Cost Optimization of Water Distribution Networks without Storage Tanks

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KEYWORDS:
Optimization, Water Distribution Networks, Genetic Algorithm, Centrifugal Pump

***Abstract*— This paper investigates the optimal design of water distribution networks without storage capacity using mathematical optimization. Both micro-genetic algorithm and hydraulic simulation of water network are used to find the optimal diameters of pipes of the network. The single-objective function of the optimization problem is the total cost of the capital costs of the piping network and the pump, and the operating cost of the pump, under the fulfillment of minimum nodal pressure requirements. An optimization methodology, Genetic Algorithm for Pumps (GAPUMPS), is used to minimize energy consumption. The described procedure is demonstrated in four case studies for water distribution networks with or without pumps. Three case studies are already established in the literature as traditional benchmark networks: the two-loop network, the three-loop network without pump, and the three-loop network with centrifugal pump. The results showed good agreement with the literature. The micro-genetic algorithm used in this study obtained better, or at least similar, optimal costs in the number of function evaluations compared with those of other evolutionary algorithms reported in the literature.**

I. INTRODUCTION

Water distribution networks (WDNs) are essential and fundamental components of any municipal infrastructure system that aim to meet residential, commercial and industrial water demands. Most water distribution networks are designed as loop systems to increase the availability of water during pipe failures in addition to increasing their hydraulic reliability and performance. Because WDNs consist of pipes, valves, pumps, and other accessories, both their capital and operating costs are huge. A major drawback of WDNs is, in specific, their high energy consumption and low efficiency. Thus, further research is needed to reduce their energy consumption and carbon footprint.

The main objective of water distribution system optimization is to find the optimal sizes and characteristics of the components of water distribution system, such as pipe diameters, pump heads and maximum power, and tanks storage volume and elevation, from the available commercial choices to provide an adequate quantity of water for drinking, domestic household usage, garden use, firefighting and irrigation. This is achieved by minimizing the operating and capital costs of these components while the constraints (minimum nodal pressure and demand) at the consumer nodes are fulfilled and the hydraulic laws (mass and energy conservations) are maintained. General recent and thorough reviews on the water distribution network (WDN) optimization can be found [1–8].

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NOMENCLATURE

C	Hazen-Williams roughness coefficient
$C_{Penalty}$	penalty function
C_{Power}	energy cost
C_{Pump}	capital cost constant of the pump
C_{PV}	series present worth factor
C_T	total cost
D	pipe diameter
$D_{\max}$	maximum available pipe diameter
$D_{\min}$	minimum available pipe diameter
E_f	energy added to the water by a pump
$f_{Pump\_1}(\cdot)$	candidate pump k capital cost
$f_{Pump\_2}(\cdot)$	annual pump operating energy cost
g	gravitational acceleration
H	head of pump
H_d	head at the operating point of pump
h_f	head loss due to friction in a pipe
H_k^{Rated}	rated head at best efficiency point of pump k
$H_{\max}$	maximum head of pump
$H_{\min}$	minimum required nodal pressure head
H_{res}	head of reservoir
$H_{sh\text{-}off}$	shut-off head of pump (the head at $Q = 0$)
Idum	initial random number seed for the GA run
ir	interest rate per year
L	length of pipe
Maxgen	maximum number of generations to run by the GA
N_{chrome}	number of chromosomes in individual's binary string
N_{Loop}	number of loops in the network
N_{Node}	number of nodes in the network
N_{op}	number of pump operating hours per year
N_{param}	number of parameters in individual's binary string
N_{Pipe}	number of pipes in the network
N_{popsiz}	population size of a GA run
N_{posibl}	array of integer number of possibilities per parameter
N_{Pump}	number of pumps in the network
NYear	project life
Q	pump flow rate
Q_d	flow rate at the operating point
Q_i	pipe flow rate in pipe i
Q_k^{Rated}	rated flow rate at best efficiency point of pump k
$Q_{\max}$	maximum flow rate of pump
Z	elevation of node
Z_{opt}	objective function

Greek symbols

α, β	constants in the Hazen-Williams friction formula
η	efficiency of pump
η_d	efficiency of pump at the operating point
$\eta_{\max}$	maximum efficiency of pump
ν	kinematic viscosity of water
ρ	density of water
ω	numerical conversion constant in the Hazen-Williams friction formula

Abbreviations

CPU	Central Processing Unit
FEN	Function Evaluation Number
GA	Genetic Algorithm
GAPUMPS	Genetic Algorithm for Pumps
WDN	Water Distribution Network
μGA	Micro-Genetic Algorithm

In order to achieve this optimal design (minimum capital and operating cost design), traditional trial and error approaches or more effective optimization methods can be used. Due to the complexity of these systems, such as different pump types, valves, reservoirs, head losses, substantial changes in pressure values, and different demand loads, the optimization procedure using trial and error approaches would be problematic in water distribution systems. As a result, numerous optimization techniques, such as linear programming gradient method, nonlinear programming methods and metaheuristic techniques, are increasingly used in water distribution system optimization procedures.

The optimization of WDNs has started since 1960s [9]. Most of the optimization approaches were based on linear and nonlinear techniques in order to find the optimal pipe diameters. The pipeline diameters were assumed to take different configurations: (a) continuous pipe diameters [10], which are not practical because of the discrete commercial diameters in the market, (b) discrete pipe diameters and split-pipe (i.e., a pipe with two different pipe diameters) [11], where the optimization variables were the lengths of these sections, and (c) discrete pipe diameters without split-pipe [12]. In the present study, the latter alternative, discrete pipe diameters without split-pipe, is investigated.

In the last decade and since the 1990s, traditional optimization techniques based on linear and nonlinear programming have been abandoned and many researchers embarked on the application of metaheuristic algorithms. Metaheuristic algorithms are optimization methods used to find optimal, or near-optimal, solution of optimization problems. Their behavior is stochastic; the optimization process is started by generating random solutions. According to Agrawal et al. [13], these algorithms make a tradeoff between the exploration phase (i.e., thoroughly investigating the promising search space) and the exploitation phase, i.e. local searching of promising area(s) discovered in the exploration phase. These metaheuristic approaches have many advantages over mathematical approaches: (a) gradient-based derivatives are not required, (b) initial vector guess is not required, and (c) ability to consider continuous variables and discrete variables without additional processing [14].

Metaheuristic algorithms are classified into two main categories [13]:

(i) Single solution based metaheuristic algorithms: they start the optimization process with one solution, and the solution is updated during the iterations. Their disadvantages are possible trapping into local optima and not exploring the search space thoroughly.

(ii) Population based metaheuristic algorithms: These algorithms generate a population of solutions that evolves over time as the number of generations or iterations increases. These algorithms are beneficial because they avoid local optima, have great exploration of search space, and have the quality of jump towards the promising part of search space. Therefore, these algorithms are used in optimizing real-world problems.

Metaheuristic algorithms can be divided into four categories based on their behavior [13]: (a) evolution-based (inspired from the natural evolution), (b) swarm intelligence-based (inspired by the social behaviors of insects, animals, fishes or birds), (c) physics-based (inspired by the rules of physics in the universe), and (d) human-related algorithms (inspired by human behavior).

Metaheuristic algorithms are used majorly in the field of water distribution networks as Genetic Algorithms [15–17], Ant Colony Optimization [18, 19], Simulated Annealing [12], Shuffled Complex Evolution [20], and Harmony Search [21], among others.

The comparison of evolutionary multiobjective optimization algorithms in the optimum design of WDNs [22] and the comparison of water distribution system design reliability (i.e., mechanical and hydraulic) using multiobjective optimization [23] are beyond the scope of the present study, which is mainly concerned with single objective optimization.

In the present study, the optimization of two types of water distribution networks is included:

A. Optimization of WDN without Pumps "Gravity Networks" - Pipe Sizing

The limited computational power in the past obliged researchers to introduce optimization to the design of WDNs without pumps for simple problems of pipe sizing. The decision variables of these problems are pipe sizes (diameters) for each pipe in the network. Given a discrete set of commercially available diameters, the pipe sizes can be selected. Accordingly, the possible discrete designs of this combinatorial optimization problem (i.e., the size of the solution space) is the number of available discrete diameter sizes to the power of the number of pipes in the network.

B. Optimization of WDN with Pumps - Energy Efficiency

The recent progress in computational power enabled researchers to tackle the optimization of WDN components (e.g., pumps, tanks and valves) and operational issues for example: pump operation and energy consumption, tank filling and emptying and valves number and locations [24]. In these problems, improving energy efficiency is the objective function. Energy efficiency includes, among others, higher water pumping efficiency. Two types of centrifugal pumps are used in WDNs: fixed-speed pumps and variable-speed pumps. Fixed-speed pumps are preferred in large WDNs, where a continuous water head is required. On the other hand, variable-speed pumps are useful in networks with varied water demand whereby the pump size is not a deciding factor [25–28]. Optimization of pumping systems can be achieved by either using direct, optimal schedule computation and optimal pump combinations, or indirect, storage trigger level, operations [29]. Design optimization of centrifugal pumps was performed to enhance the pump performance [30, 31].

The main aim of the present study is to develop a simulation procedure (code) to optimize the previously mentioned types of water distribution networks. The code links the genetic algorithm with the Newton-Raphson method for hydraulic simulation of water networks. Four case studies with different levels of complexity are used to examine the code's applicability.

The remaining part of this paper consists of six sections. Section 2 presents the governing equations of water pipeline system optimization. Section 3 is devoted to the explanation of the performance curves of constant speed centrifugal pumps. Section 4 introduces some fundamental aspects of genetic algorithm (GA). Section 5 illustrates the case studies used in the present study. The results and the discussion of these case studies are presented in Section 6. Finally, the conclusions of the paper are settled in Section 7.

II. OPTIMIZATION OF PIPELINE SYSTEMS

Water distribution network design is formulated as a least-cost optimization problem with the selection of pipe diameters and pump size as the decision variables. The pipes layout, nodal demands, and minimum nodal heads requirement are assumed to be known in the case of WDN without pumps, whereas the pipes layout, pipes diameters, nodal demands, and minimum nodal heads requirement are assumed known in the case of WDN with pumps.

A. Decision Variables

The decision variables are the required intervention actions and depend on the problem under investigation. Due to the available discrete commercial pipe diameters in the market, discrete decision variables are more practical than continuous decision variables for pipe diameters in the optimization of water distribution networks [8]. For optimization of a new water distribution system, the decision variables are the pipe diameters and the pump configuration; whereas for rehabilitation, the decision variables include replacement, relining, or no action for the pipe.

The total number of the decision variables depends on the water distribution system condition, i.e. gravity network or network with a pump:

(a) The total number of the decision variables is the number of pipes for a gravity network (i.e., without pump).

(b) The total number of the decision variables is the number of pipes and the pump characteristics (i.e. pump sizes) for a network with a pump.

B. Constraints

The objective function is to be minimized under two major constraints, namely the model constraints (conservation of mass and energy) and the design and hydraulic constraints (available pipe diameters and minimum pressure requirements).

(a) Model constraints:

For the steady state, the conservation of mass reads:

$$\sum_{j=1}^{N_{Node}} Q_j = 0 \tag{1}$$

where Q_j is the discharges into or out of the node j (sign included) and N_{Node} is the total number of nodes in the network.

The conservation of energy states that the total head loss around any loop must equal to zero or is equal to the energy added to the water by a pump in loop l, E_{pl}, if there is any:

$$\sum h_{fi} = E_{pl} \qquad l = 1,...,N_{Loop} \tag{2}$$

where h_{fi} is the head loss due to friction in a pipe i in loop l (i is the pipe number to be summed up in the loop l) and N_{Loop} the total number of loops in the network. The friction head loss in pipe i, h_{fi}, is expressed in the present study by the Hazen-Williams friction formula [15]:

$$h_{fi} = \omega \frac{L_i}{C_i^{\alpha} D_i^{\beta}} Q_i^{\alpha} \tag{3}$$

where Q_i is the pipe flow rate in pipe i, L_i the length of pipe i, D_i the diameter of pipe i, C_i the Hazen-Williams roughness coefficient, α and β are constants, and ω is a numerical conversion constant, which depends on the units used. The values of α, β and ω are known for in each case in the present study.

(b) Design and hydraulic constraints:

The design constraints (the pipe diameter bounds) and the hydraulic constraints (the nodal pressure head bounds) are given respectively as:

$$D_{\min} \le D_i \le D_{\max} \qquad i = 1,...,N_{Pipe} \tag{4}$$

$$H_j \ge H_{j,\min} \qquad j = 1,...,N_{Node} \tag{5}$$

where D_i is the discrete diameter of pipe i, which is selected from the set of commercially available pipe diameters, H_j is the pressure head at node j, $H_{j,\min}$ is the minimum required pressure head at node j, and N_{Pipe} is the total number of pipes in the network.

C. Objective Function

The value of the objective function, Z_{opt}, is positive for maximization and negative for minimization. In the present study, this value depends on the studied cases as:

(a) Network without pump: when the total cost of the given network is minimized, the total cost, C_T, (i.e., the pipes cost) and the objective function, Z_{opt}, are calculated as:

$$C_T = \sum_{i=1}^{N_{Pipe}} c_i\left(D_i\right) \cdot L_i \tag{6}$$

$$Z_{opt} = \begin{cases} -C_T & \text{if} \quad H_{j,\min} - H_j \le 0 \\ -C_T\left[1 + \dfrac{1}{N_{Node}} \displaystyle\sum_{j=1}^{N_{Node}} \left(H_{j,\min} - H_j\right)\right] & \text{else} \end{cases} \tag{7}$$

where $c_i(D_i)$ is the cost of commercially available pipe i, whose diameter is D_i, per unit length. The unit cost $c_i(D_i)$ is generally a non-linear function of diameter. The second term in the second formulation of Eq. (7) is the penalty function [32] which is based on the pressure head violation as given by Eq. (5):

$$C_{Penalty} = \frac{C_T}{N_{Node}} \sum_{j=1}^{N_{Node}} \left(H_{j,\min} - H_j\right) \tag{8}$$

The aim of introducing the penalty cost function is to prevent the genetic algorithm from searching in the infeasible solution area.

(b) Network with a pump: In this case, the total cost, C_T, which is the life cycle cost, is minimized. It is the sum of the capital cost of the pipes and the pump, and the operating cost of the pump. It is calculated as follows [33]:

$$C_T = \sum_{i=1}^{N_{Pipe}} \left[c_i\left(D_i\right) \cdot L_i\right] + \sum_{k=1}^{N_{Pump}} \left[f_{Pump\_1}\left(Q_k^{Rated}, H_k^{Rated}\right) + C_{PV} \cdot f_{Pump\_2}\left(Q_k, H_k, \eta_k\right)\right] \tag{9}$$

The objective function, Z_{opt}, was previously calculated by Eq. (7). In Eq. (9), N_{Pump} is the number of pumps in the network, and $f_{Pump\_1}(Q_k^{Rated}, H_k^{Rated})$ is the capital cost of the candidate pump k, which is calculated as follows [34]:

$$f_{Pump\_1}\left(Q_k^{Rated}, H_k^{Rated}\right) = C_{Pump} \cdot \left(Q_k^{Rated}\right)^{0.7} \cdot \left(H_k^{Rated}\right)^{0.4} \tag{10}$$

where C_{Pump} is the capital cost constant of the pump and Q_k^{Rated} (m^3/s) and H_k^{Rated} (m) are the rated flow rate and head at the best efficiency point of the pump k, respectively. It is known that at the maximum efficiency, the differentiation of the pump efficiency equation is equal to zero. Consequently, the Q_k^{Rated} is determined. The rated pump head H_k^{Rated} can be calculated by substituting with Q_k^{Rated} (m^3/s) in the pump head equation for pump k [35]:

$$H_k\left(Q_k\right) = a + bQ_k + cQ_k^2 \tag{11}$$

where a, b, and c are the polynomial coefficients representing the pump head quadratic equation.

In Eq. (9), the last term on the right-hand side is the pump operating cost. C_{PV} is the series present worth factor, which is equal to the present amount of money divided by the annual amount of money, and is calculated as [35]:

$$C_{PV} = \frac{\left(1+ir\right)^{NYear} - 1}{ir\left(1+ir\right)^{NYear}} \tag{12}$$

where ir is the interest rate per year, and $NYear$ is the project life span.

The annual pump operating energy cost $f_{Pump\_2}(\cdot)$, Eq. (9), is obtained as follows:

$$f_{Pump\_2}\left(Q_k, H_k, \eta_k\right) = \frac{\rho g Q_k H_k}{\eta_k} \cdot \frac{N_{op}}{10^3} \cdot C_{Power} \quad (13)$$

where ρ is the density of water, g is the acceleration of gravity, Q_k, H_k and η_k are the discharge, head and efficiency of each candidate pump k, respectively, N_{op} is the number of pump operating hours per year which is at 8760 in the present study, and C_{Power} is the energy cost. All economic data are extracted from Walski et al. [34].

III. Performance Curves of Constant Speed Centrifugal Pumps

In order to pump water into the whole water distribution system, pumping stations consist of pumps with various capacities. As previously mentioned, fixed speed pumps or variable speed pumps may be used according to the design of the WDN. Case 4 in the present study utilizes a fixed speed pump because it requires fixed nodal demands. This section is devoted to evaluate the power consumption of the centrifugal pumps in this case, and to compare between this method and the traditional one used by Costa et al. [33].

The optimal head and efficiency curves of centrifugal pumps (*H-Q* and *η-Q*) are obtained for fixed speed pumps. To approximate the *H-Q* and *η-Q* curves, the quadratic equations (14) and (15) are used for fixed speed pumps [36, 37]. However, the numerical value of the coefficient b is assumed to be zero in order to avoid double operating points. This means that the shut-off head, $H_{sh\text{-}off}$, is the maximum head value, $H_{\max}$, in the *H-Q* curve. These two equations are the general quadratic equations:

$$H = a + bQ + cQ^2 \quad (14)$$

$$\eta = d + eQ + fQ^2 \quad (15)$$

where a, b, c, d, e and f are the coefficients which determine the shape of the curves. Figure 1 shows the relationship between the *H-Q* curve and the *η-Q* curve at the operating point, d, ($H = H_d$, $Q = Q_d$) and the design point ($\eta = \eta_{\max}$, $Q = Q^{Rated}$). In addition, the 'Run Out' point (i.e., pump's actual maximum flow, Q_{Runout}) and the maximum discharge, $Q_{\max}$, (i.e., extension of pump's characteristic and efficiency curves at $H = 0$ and $\eta = 0$, respectively) are also shown in the same figure.

The coefficient a in Eq. (14) is obtained by setting the head (H) equal to $H_{\max}$ at zero flow rate ($Q = 0$). Then, Eq. (14) becomes:

$$a = H_{\max} \quad (16)$$

In the same manner, by setting the efficiency equal to zero ($\eta = 0$) at zero flow rate ($Q = 0$) in Eq. (15), one obtains the value of coefficient d from Eq. (15):

$$d = 0 \quad (17)$$

The remaining coefficients of the quadratic equations (14) and (15), c and e, are obtained by setting the values of H and η equal to zero. Therefore, the flow rate at this point assumes a maximum value $Q_{\max}$, see Fig. 1.

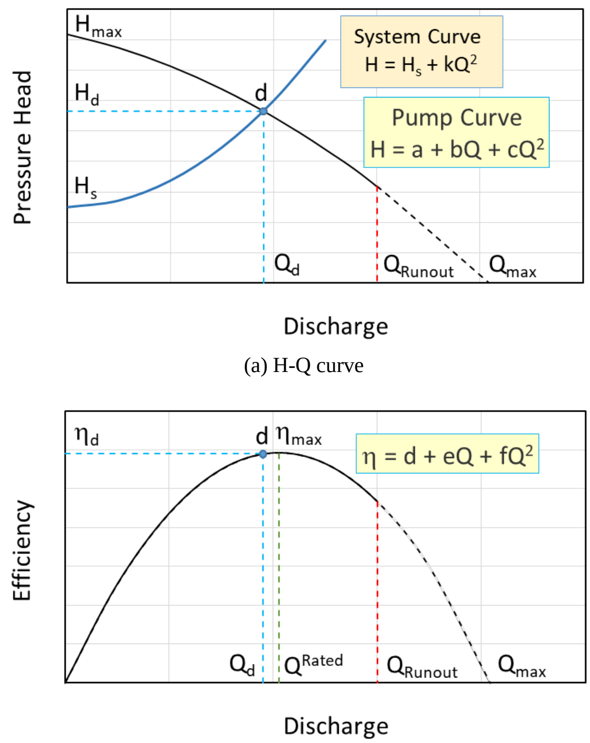
$$c = -\left(H_{\max} + bQ_{\max}\right)/Q_{\max}^2 \quad (18)$$

$$e = -f\,Q_{\max} \quad (19)$$

Furthermore, in order to deduce the relationship for the maximum efficiency, the first derivative of equation (15) is set at zero as:

$$d\eta/dQ = 2fQ^{Rated} + e = 0 \quad (20)$$

$$Q^{Rated} = -0.5e/f \quad (21)$$



(a) H-Q curve

(b) η-Q curve

Fig. 1. Characteristic and efficiency curves of centrifugal pump

Using equations (15), (17), and (21), the maximum efficiency is:

$$\eta_{\max} = -0.25e^2/f \quad (22)$$

From equations (19) and (22), the coefficient f is calculated as:

$$f = -4\eta_{\max}/Q_{\max}^2 \quad (23)$$

The pump power is obtained using the pump head and efficiency defined in Eqs. (14) and (15), respectively, as:

$$Power = \frac{\gamma Q_d H_d}{\eta_d} = \frac{\gamma Q_d\left(a + bQ_d + cQ_d^2\right)}{d + eQ_d + fQ_d^2} \quad (24)$$

where Q_d, H_d, and η_d are the discharge, head and efficiency at the operating point, respectively, see Fig. 1. Substituting Eqs. (16), (18), (17), (19) and (23), which define the coefficients a, c, d, e and f, respectively, gives, after some mathematical manipulations, the following expression for pump power:

$$Power = 0.25\frac{\gamma}{\eta_{\max}}\left[H_{\max}\left(Q_{\max}+Q_d\right)+b\,Q_{\max}\,Q_d\right] \quad (25)$$

The linear relationship of the centrifugal pump power with the operating point flow rate, Q_d, and the variables H_{max}, Q_{max}, and η_{max}, which are determined from Eqs. (16), (19), and (22), shows that the minimum centrifugal pump power corresponds to the minimum demands at the nodes. Furthermore, as previously mentioned, the double operating points are avoided by setting the coefficient b equal to zero, as suggested [36, 37]. Accordingly, the centrifugal pump power, Eq. (25), becomes:

$$Power = 0.25\gamma\frac{H_{\max}}{\eta_{\max}}\left(Q_{\max}+Q_d\right) \quad (26)$$

The centrifugal pump power at shut-off (Q = 0) is thus:

$$Power_{shut-off} = 0.25\gamma\frac{H_{\max}}{\eta_{\max}}Q_{\max} \quad (27)$$

In addition, the essential condition of $dPower/dQ_d > 0$ of Eq. (25) for the linear relationship of the pump power with the operating point flow rate, Q_d, gives: $-b < H_{max}/Q_{max}$, i.e. the coefficient $-b$ must be less than the ratio H_{max}/Q_{max}.

This simplified evaluation of the centrifugal pump power presented in Eq. (25) has two drawbacks: (a) the pump discharge flow, Q_d, is used in the derived equation and is not known: in some networks without tanks and one pump only, Q_d equals to the total nodal demands, and (b) the linear relationship of the centrifugal pump power with the operating point flow rate, Q_d, is not very accurate: the actual relationship is a third-order polynomial equation or higher [38]. On the other hand, despite the drawbacks of Eq. (25), the power calculation of centrifugal pump is simplified as the only required unknown is the pump discharge flow, Q_d, after estimating the maximum discharge, Q_{max}, maximum head, H_{max}, maximum efficiency, η_{max}, and coefficient b, which are fixed for the specified centrifugal pump. The evaluation of the head, H_d, and efficiency, η_d, at the operating point are not used in Eq. (25).

IV. GENETIC ALGORITHM (GA)

Genetic algorithms (GAs) are nature behavior based incidental computational techniques which mimic the evolution theory of Charles Darwin. GAs have dramatic popularity among other evolutionary algorithms because of their flexibility, adaptability, robustness and proven usefulness across a wide range of optimization problems in engineering, science and commerce [39]. GAs are based on the biological evolutionary principle of survival of the fittest, which tries to hold genetic data from old generations (parents) to new generations (children). The GA has the ability to solve linear, nonlinear, nonconvex, multimodal, continuous and discrete problems. Thus, it is applicable in pipe network optimization problems especially under unsteady cases.

Figure 2 shows a simple GA cycle. This GA cycle steps are:

(a) Initialization: generation of initial population (chromosomes) of candidate solutions,
(b) Selection: selection of a pool of parents from the parent population based on the fitness values,
(c) Crossover: taking pairs of parents from the parent population and swapping their genetic information between them to produce children,
(d) Mutation: introducing "new" genetic material into the population by randomly changing codons on the chromosome between 0 and 1,
(e) Evaluation: assigning a fitness value to each child in order to rank a population,
(f) Replacement: replacing the previous population with the newly generated child population,
(g) Termination: returning the fittest solution in the population 'the best solution' after a predefined number of generations or other constraint on the runs.

Simple GA procedures use population of large size in the range of 30 to 200 individuals [39]. These procedures require long computations. In the present study, however, the fitness function after each generation was obtained using a small population size GA referred to as micro-Genetic Algorithm (μGA) [40] in order to reduce the time penalty.

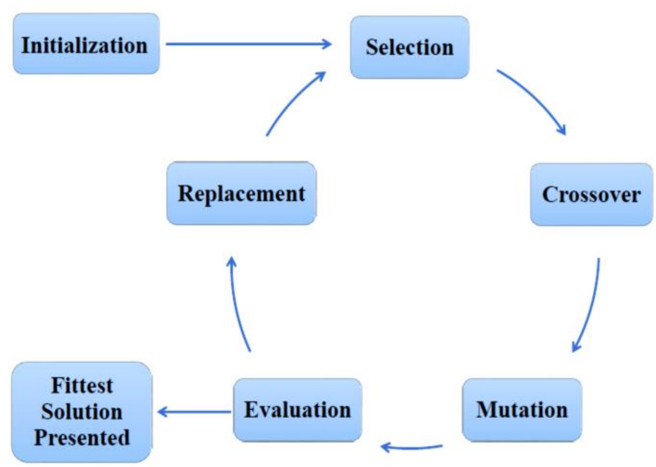


Fig. 2. GA cycle

In order to obtain the optimal pipe network design, an optimization code is developed in the present study using FORTRAN. The developed code GAPUMPS (Genetic Algorithm for Pumps) links the GA developed by Carroll [41] with Newton-Raphson simulation technique for the hydraulics of WDNs. The flow chart of this code is illustrated in Fig. 3. This code follows the steps proposed by Simpson et al. [42] in using GA for pipe network optimization as follows:

1. Generation of initial population of coded strings of population size N_{popsiz}: each N_{popsiz} population string represents a possible combination of pipe diameters.
2. Computing the cost of the network for each of N_{popsiz} solutions after changing the generated pipe sizes to the corresponding commercially-available pipe sizes.
3. Performing steady state hydraulic simulations, using Newton-Raphson method, of each network, and finding the

nodal pressure heads and pipes flow rates for each population under the required nodal demands.

4. Computing the penalty cost after comparing the calculated nodal pressure heads, H_j, with the minimum allowable pressure heads, $H_{j,\min}$, and assigning a penalty cost if the nodal minimum pressure constraints are not fulfilled. The penalty cost is computed by Eq. (8) where the penalty factor is C_T / N_{Node} [8].
5. Computing the total network cost of each design in the current population, which is the sum of the network cost (Step 2) and the penalty cost (Step 4).

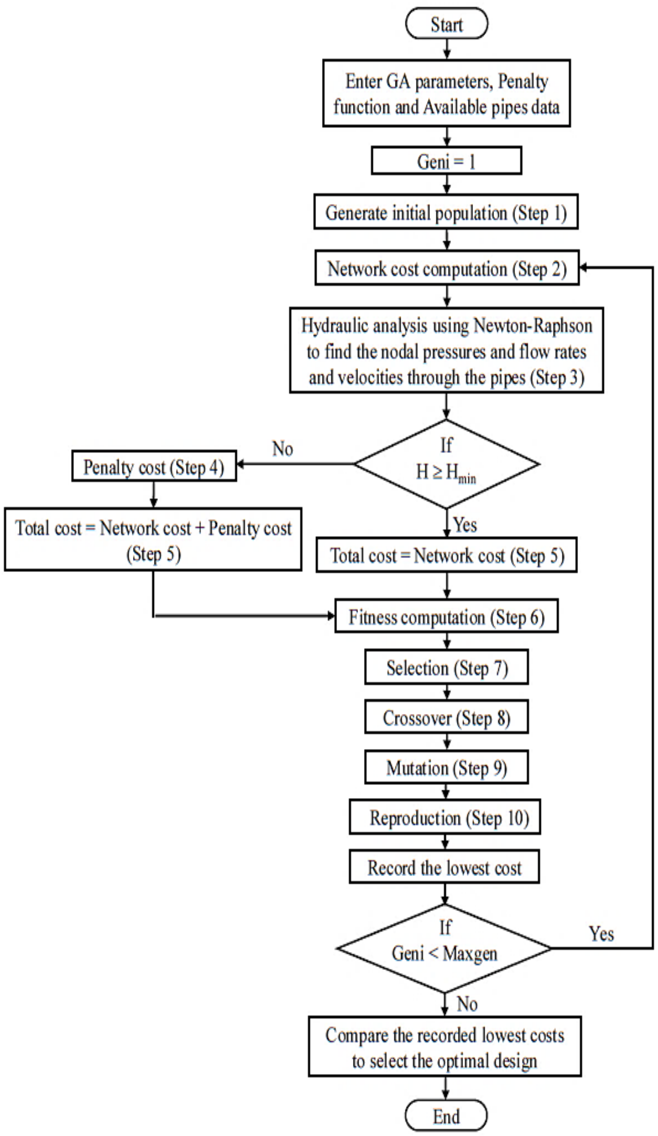


Fig. 3. Flow chart of the GAPUMPS code

6. Computing the fitness function, Eq. (7), which is a measure of the GA generated solutions quality. The fitness function is taken as the negative value of the total network cost.
   The following genetic operators are used to generate new population.
7. The selection operator: the fitness function gives a rank for each string in the population from which the GA selects the parents of the next generation. The classic roulette wheel scheme is the used selection method.
8. The crossover operator. Each selected pair of parents from Step 7 undergoes a crossover operation with some specified probability according to the crossover technique. The uniform crossover of 0.5 is used [43].
9. The mutation operator. Mutation occurs with some specified mutation probability for each bit in the population strings. The mutation probability is set to be $(N_{chrome}/N_{param})/ N_{popsiz}$ or $1/N_{popsiz}$ for the creep mutation or jump mutation, respectively, which are applied in the present study.
10. Production of successive generations. A new offspring is generated through Steps 7-9, then the lowest cost design is recorded and Steps 2-9 are repeated until the maximum number of generations is reached. Finally, the GA compares between the recorded lowest costs and selects the optimal pipe network design.

The used genetic algorithm code by Carroll [41] provides the following additional parameters to obtain the global solution: *Idum*: the initial random number seed (equal to negative integer) for the GA run, *Maxgen*: the maximum number of generations to run by the GA (*Maxgen* = 200), N_{posibl}: the array of integer number of possibilities per parameter, and N_{popsiz} = 12. The GA investigates the *Idum* and N_{posibl} which provide the optimal solution with the least time consumption.

V. Case Studies

In this paper, the performance of Newton-Raphson method for hydraulic simulation of WDNs combined with optimization using GA is demonstrated in four case studies using two WDN configurations. Two case studies are two-loop networks and the other two case studies are three-loop networks. However, the number of pipes and nodes in these networks do not represent a practical type of networks in the real life, both network configurations are benchmark problems in WDN optimization, [33], [44]. Thus, the present study gives good indication about the validity of the GAPUMPS code in obtaining the optimal cost. According to [44], they first validated their method using the two-loop network and then implied the same method in analyzing a more complex network, which would be done in the future work. Table I shows the parameters of the four case studies, namely the number of pipes, nodes, reservoirs, and pumps. Table II shows the commercially-available pipe sizes and cost per meter for Cases 1-4.

TABLE I
Case Studies Data

Case Study	**No. of Pipes N_{Pipe}**	**No. of Nodes N_{Node}**	**No. of Reservoirs**	**No. of Pumps N_{Pump}**	**Water Level in Reservoirs**
Case 1: Two-loop, (Ref. [44])	8	7	1	--	H_{res1} = 210 m
Case 2: Two-loop with additional reservoir	9	8	2	--	H_{res1} = 210 m H_{res2} = 150 - 550 m
Case 3: Three-loop without pump, (Ref. [33])	11	9	1	--	H_{res1} = 235 m
Case 4: Three-loop with pump, (Ref. [33])	11	9	1	1	H_{res1} = 190 m

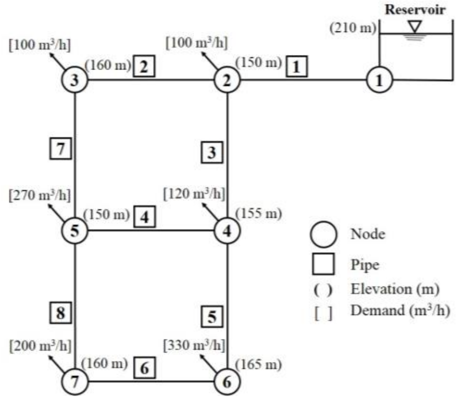


Fig. 4. Original two-loop network [44] - Case 1

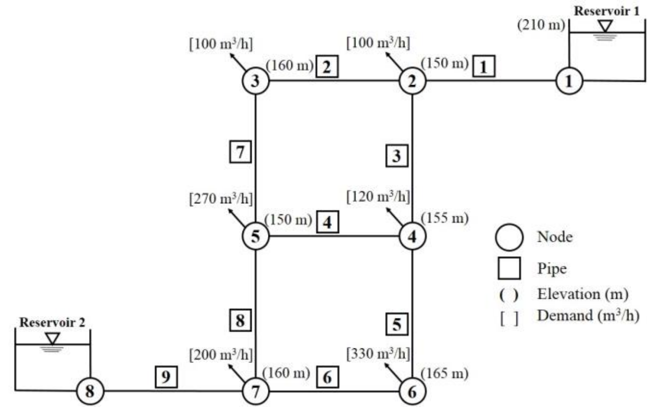


Fig. 5. Two-loop network with additional reservoir 2 (Reservoir 2 water level is variable) - Case 2

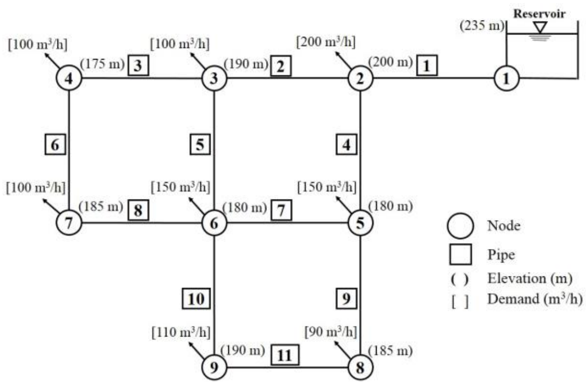


Fig. 6. Three-loop network without pump [33] - Case 3

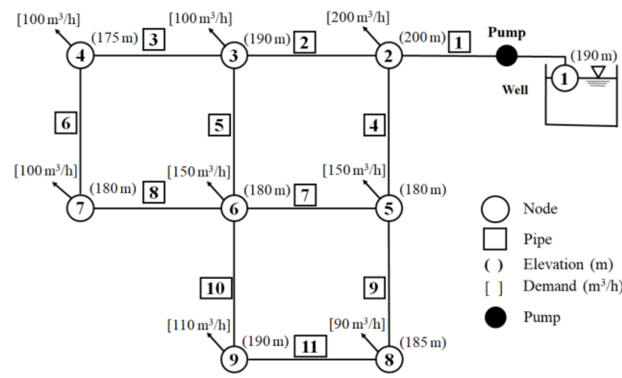


Fig. 7. Three-loop network with pump [33] - Case 4

TABLE II
COMMERCIALLY AVAILABLE PIPE SIZES AND COST PER METER FOR CASES 1-4

Cases 1 and 2			**Cases 3 and 4**		
D (in.)	**D (mm)**	**Cost (units/m)**	**D (in.)**	**D (mm)**	**Cost ($/m)**
1	25.4	2	6	152.4	42.0
2	50.8	5	8	203.2	58.4
3	76.2	8	10	254.0	73.8
4	101.6	11	12	304.8	95.8
6	152.4	16	14	355.6	118.8
8	203.2	23	16	406.4	143.0
10	254.0	32	18	457.2	169.0
12	304.8	50	20	508.0	197.2
14	355.6	60	24	609.6	252.6
16	406.4	90	30	762.0	346.1
18	457.2	130			
20	508.0	170			
22	558.8	300			
24	609.6	550			

A. *Case 1: Two-loop network [44]*

This two-loop network is a well-known and small-size water distribution network, which was used for the first time in the literature by Alperovits and Shamir [44]. It was subsequently used by many investigators [11, 12, 15, 45–50] in order to study the optimization process of WDNs. It consists of a constant head reservoir (210 m), eight pipes, each of which has a length of 1000 m, and seven nodes, see Figure 4. The reservoir, whose elevation is 210 m, is the water source used to supply the demand of each node. The details of this network are given in Table III. The nodal pressure heads are required to be higher than their elevation by at least 30 m. For all pipes of this network, the Hazen-Williams coefficient C is taken as 130.

The cost data proposed by Alperovits and Shamir [44] for the 14 commercially-available pipe diameters in Cases 1 and 2 are given in Table II. The mixed units (SI and foot-pound-second) are used in this study since they were applied in Alperovits and Shamir [44] and other previously mentioned researchers optimizing this network [11, 12, 15, 45–50]. In the present study, the parameters of the Hazen-Williams equation, Eq. (3), are $\alpha = 1.852$, $\beta = 4.8704$, and $\omega = 10.6744$ (the units of Q are m^3/s and D and L are m) [32].

B. *Case 2: Two-loop network with additional reservoir 2 whose water level is variable (Present study)*

As shown in Figure 5, a modified configuration of the two-loop network shown in Case 1 is developed by adding another reservoir. This developed network is formed by linking reservoir 2 to node 7 in Case 1 via pipe 9. The nodes and pipes data are presented in Table III. The effect of the water level in reservoir 2 on the network optimization is studied in this case. The water level of reservoir 2 is modified in the range from 150 m to 550 m. The parameters of the Hazen-Williams equation, Eq. (3), for the commercial pipes are $\omega = 10.6744$,

$\alpha = 1.852$, $\beta = 4.8704$, and $C = 130$ as in Case 1. The cost data are similar to those provided for Case 1, Table II.

TABLE III
PIPE AND NODE DATA OF THE THREE-LOOP WATER NETWORK, CASES 1-2

Pipe	From Node	To Node	L (m)	Friction C	Node	Demand (m^3/h)	Z (m)
1	1	2	1000	130	1	--	210
2	2	3	1000	130	2	100	150
3	2	4	1000	130	3	100	160
4	4	5	1000	130	4	120	155
5	4	6	1000	130	5	270	150
6	7	6	1000	130	6	330	165
7	3	5	1000	130	7	200	160
8	7	5	1000	130	8	--	--*
9	8	7	100	130			

- Case 1: Pipe 1-8 and nodes 1-7. - Case 2: Pipe 1-9 and nodes 1-8.
* Different heads are supposed.

C. *Case 3: Three-loop network without pump [33]*

The water distribution network shown in Figure 6 was developed by Costa et al. [33]. This network consists of a constant head reservoir (235 m), eleven pipes, each of which has a length 2500 m, and nine nodes. The reservoir is used as a source to supply the demand of each node. Its elevation is 235 m, as illustrated in Figure 6. The pressure heads at all nodes are required to be higher than their elevations by at least 30 m.

The available diameters and the corresponding cost per meter of 10 commercial pipes for Case 3 are listed in Table II. The parameters of the Hazen-Williams equation, Eq. (3), for the commercial pipes are $\omega = 10.5088$, $\alpha = 1.85$, $\beta = 4.87$, and $C = 130$ [33]. The data of the pipes and nodes of the network are given in Table IV.

TABLE IV
PIPE AND NODE DATA OF THE THREE-LOOP WATER NETWORK, CASES 3-4

Pipe	From Node	To Node	L (m)	Friction C	Node	Demand (m^3/h)	Z (m)
1	1	2	2500	130	1	--	235
2	2	3	2500	130	2	200	200
3	3	4	2500	130	3	100	190
4	2	5	2500	130	4	100	175
5	3	6	2500	130	5	150	180
6	4	7	2500	130	6	150	180
7	5	6	2500	130	7	100	185
8	6	7	2500	130	8	90	185
9	5	8	2500	130	9	110	190
10	6	9	2500	130			
11	8	9	2500	130			

D. *Case 4: Three-loop network with one fixed speed pump [33]*

This case is similar to Case 3 with an additional feed pump located between the 190 m constant head reservoir and node 1 as shown in Figure 7. This network was developed by Costa et al. [33] and further investigated in [35, 51]. The lengths of all the pipes of this network are 2500 m with the same Hazen-Williams parameters as Case 3. In addition, the costs per meter of the available commercial pipe diameters in Case 4 are listed in Table II. The demand of each node at its elevation is illustrated in Figure 7 and Table IV. The pressure heads at all nodes are required to be higher than their elevations by at least 30 m.

The coefficients of the pump head quadratic equation *a*, *b*, and *c*, Eq. (11), are listed for each pump in Table V. Ten candidate pumps are considered in the optimization process. The first candidate pump (number 1) corresponds to a "no pump" option.

The capital cost constant of the pump C_{Pump} is 700,743 \$, Eq. (10). A single pump efficiency curve is used for all pumps. The pump efficiency equation employed in the current study is [33, 35, 51]:

$$(\%) \quad \eta_k = -695.4 Q_k^2 + 418.3 Q_k + 2.857 \tag{28}$$

$$d\eta_k / dQ_k = -1390.8 Q_k + 418.3 = 0 \tag{29}$$

where the units of the flow rate Q_k are m^3/s. Hence, from Eq. (29), $Q_k^{Rated} = 0.30076$ m^3/s or 1082.7437 m^3/h.

In order to evaluate the pump operating cost and annual pump operating energy cost, equations (12) and (13) are used. The interest rate per year $ir = 12\%$ and the project life $NYear = 20$ years in the present study, Eq. (12). The energy cost C_{Power} = \$0.12/kWh, Eq. (13).

TABLE V
CANDIDATE PUMP HEAD EQUATION COEFFICIENTS, [35], CASE 4

Candidate Pump Number	Coefficients *a*, *b*, and *c*, Eq. (11)		
	a	***b***	***c***
1	0	0	0
2	−72.0	−24.0	48.0
3	−81.0	−27.0	54.0
4	−125.1	10.9	55.1
5	−126.0	−9.0	65.0
6	−89.1	1.2	67.0
7	−103.7	−7.2	75.0
8	−129.6	0	79.0
9	−162.0	9.0	85.0
10	−136.7	−3.45	91.5

VI. RESULTS AND DISCUSSION

An in-house code, GAPUMPS, was used to perform hydraulic network simulation and optimization. GAPUMPS is developed using FORTRAN. It employs a single-objective optimization tool in order to minimize the total cost of a given network, Eq. (6). It can also be used to minimize life cycle cost, Eq. (9).

In this paper, GAPUMPS simulation results are verified by analyzing the hydraulic simulation of the previous cases. Subsequently, micro-genetic optimization is applied to the previously-described case studies.

In all the following cases, the density of water is ρ = 1,000 kg/m^3 and its kinematic viscosity is $\nu = 1 \times 10^{-6}$ m^2/s. The gravitational acceleration g = 9.81 m/s^2. The Hazen-Williams coefficient C for all pipes in the network is equal to 130.

TABLE VI
DATA, NODAL HEADS AND FLOW RATES FOR THE SIMULATION OF CASES 1 AND 2

			Node							
	Pipe	1	2	3	4	5	6	7	8	9
Data	Length (m)	1000	1000	1000	1000	1000	1000	1000	1000	100
	Elevation (m)		150	160	155	150	165	160		
	Demand (m^3/h)		100	100	120	270	330	200		
Case 1	Optimal diameter, m (in.)	0.4572 (18)	0.2540 (10)	0.4064 (16)	0.1016 (4)	0.4064 (16)	0.2540 (10)	0.2540 (10)	0.0254 (1)	
	Head (m) - EPANET		53.25	30.46	43.45	33.80	30.44	30.55		
	Head (m) - GAPUMPS		53.25	30.46	43.45	33.80	30.44	30.55		
	Flow rate (m^3/h) - EPANET	1120.00	336.87	683.13	32.57	530.56	−200.56	236.87	−0.56	
	Flow rate (m^3/h) - GAPUMPS	1120.00	336.87	683.13	32.57	530.56	−200.56	236.87	−0.56	
Case 2 - Reservoir 2, H_{res2} = 180 m	Optimal diameter, m (in.)	0.4572 (18)	0.2540 (10)	0.4064 (16)	0.1016 (4)	0.4064 (16)	0.2540 (10)	0.2540 (10)	0.0254 (1)	0.0254 (1)
	Head (m) - EPANET		53.22	30.43	43.39	33.77	30.36	30.36		
	Head (m) - GAPUMPS		53.22	30.43	43.39	33.77	30.36	30.35		
	Flow rate (m^3/h) - EPANET	1122.44	336.92	685.53	32.53	532.99	−202.99	236.92	0.55	−2.44
	Flow rate (m^3/h) - GAPUMPS	1122.445	336.910	685.534	32.537	532.997	−202.997	236.910	0.552	−2.445

The negative sign in flow rates means the contrary of supposed flow direction, Table III.

A. Hydraulic Simulation

Our hydraulic simulation code is verified by comparing its predictions with theose of EPANET software [52, 53]. EPANET is a hydraulic analysis code developed by the USA Environmental Protection Agency. It is capable of simulating both simple and complex networks. Thus, it has been used in a wide range of applications, regardless of the size of the network [54–56].

Table VI shows the head and flow rates calculated by the EPANET and the GAPUMPS codes for Cases 1 and 2. The input pipe diameters are the optimal diameters of the present study. These diameters are obtained from the optimization of each case under the constraint that the minimum acceptable pressure head requirements for nodes 2 to 7 must be greater than 30 m above ground level. The total discharge Q = 1120 m^3/h from reservoir 1 is the sum of the nodal demands in Case 1. The water level in reservoir 2 is 150 m in Case 2. The results are almost identical, revealing the reliability of the proposed GAPUMPS code.

B. Network Optimization

The previous optimization model of the micro-genetic algorithm is applied to the four case studies of WDN. The total number of evaluations used in all calculations is 2400, which is obtained by multiplying a population size 12 by a maximum number of generations 200. The GAPUMPS code uses Windows 7 operating system on an Intel i7-3612QM CPU@2.10 GHz for computation. Different initial random number seeds, *Idum*, are performed.

1. Network Optimization of Case 1

Although the simplicity of the studied networks, it should be mentioned that, for the simple two-loop network with eight pipes and a set of 14 commercial pipes, the total number of designs (i.e. the search space) is $14^8 = 1.48 \times 10^9$. Therefore, it is very difficult for any mathematical model to test all these possible combinations of designs and a very small percentage of combinations can be reached. As will be seen, the present function evaluation number, FEN, is 741. Therefore, the fraction of the total search space searched is 5.02×10^{-7}, which is a very small fraction of all possible designs. This shows the importance of using optimization to reach the optimal solution.

The output of various runs for the first case of the computer program reveals the optimal diameters given in Table VI. The optimal cost is 419,000 units and the constraints given in Case 1 ($H \geq 30$ m) are fulfilled. The optimal solution is obtained from the set of diameters of 18, 10, 16, 4, 16, 10, 10 and 1 inch for the links 1 to 8, respectively. It can be noticed that this optimal solution is identical to that obtained by Savic and Walters [15] and Cunha and Sousa [12]. The optimal solution obtained by applying the genetic algorithm proposed in the present study is in accordance with the optimal cost found in previous studies [1, 8, 57].

The GAPUMPS simulation results for the cost evolution of Case 1 are shown in Figure 8. The optimal solution (419,000 units) is found after a function evaluation number, FEN, of 741 for a total number of evaluations of 2400. A rapid decrease in the cost is achieved in the first 100 function evaluations and a slow decrease thereafter, a similar result to that obtained by Djebedjian et al. [32]. The performance evaluation of the optimization algorithms for the same network shows that the present function evaluation number, FEN, is 741 [8]. Thus, using micro-genetic algorithm is better than other genetic algorithms in obtaining the optimal result (e.g., Afshar [58], “FEN = 3000”; El-Ghandour and Elbeltagi [59], “FEN = 6060”; Poojitha et al. [60], “FEN = 4200”). In this paper, the GAPUMPS code takes 0.62 seconds of CPU time for a total number of evaluations of 2400.

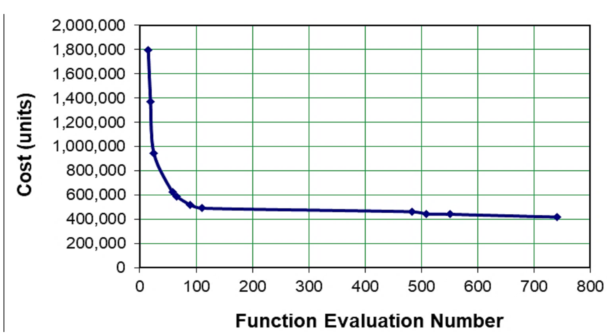


Fig. 8. Cost evolution resulting from the GAPUMPS code for Case 1

2. *Network Optimization of Case 2*

The second network consists of nine pipes and a set of 14 commercial pipes. Its total number of possible designs is $14^9 = 2.066 \times 10^{10}$, which constitutes the search space of the problem. The application of a search algorithm to this simple network would require a considerable amount of time to navigate the entire search space of potential solutions.

The output of various runs of GAPUMPS for Case 2 when the water level in reservoir 2 is 180 m is given in Table VI. The optimal cost is 419,200 units and the solution satisfies the constraints given in the problem. The diameters are greater than 1 inch and the pressure heads at the nodes are not below 30 meters. Figure 9 shows the cost evolution resulted from the GAPUMPS code for Z = 180 m. The optimal solution (419,200 units) is found after 981 hydraulic analyses for a total number of evaluations of 2400. There is a rapid convergence tendency in the first 235 function evaluation number. A CPU run time for the GAPUMPS code of 0.84 seconds is executed for 2400 function evaluations.

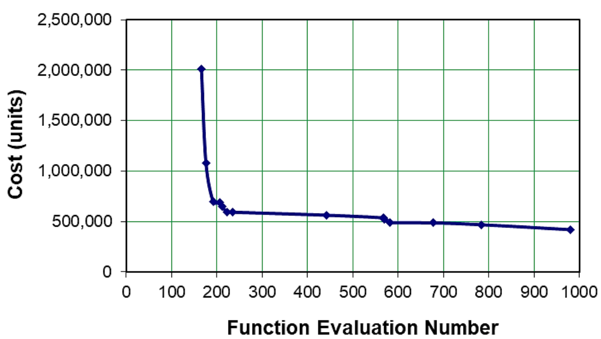


Fig. 9. Cost evolution resulting from the GAPUMPS code for Case 2 and water level in reservoir 2 = 180 m

Before analyzing the results of Case 2, it is interesting to highlight the following points:

(a) The GAPUMPS code attempts to decrease the optimal cost by selecting appropriate optimal diameters that fulfill the minimum nodal head coefficient. In achieving this task, sometimes the code may select a pipe diameter that is not practical for the network (i.e., minimum available pipe diameter $D_{\min}$), but it is suitable from the hydraulic viewpoint, e.g. a pipe is connected to a reservoir with a very small diameter as 1 inch as observed in Case 2.

(b) In Case 1, the optimal cost is a function of the optimal diameters according to Eq. (6). In Case 2, however, the optimal cost is a function of the optimal diameters and the water level in reservoir 2. This level is responsible for the flow rate feeding the network.

(c) The critical node—the node with the minimum nodal pressure head—in the most optimal designs of the network, Table VII, is node 6 or node 7 for water level in reservoir 2 less than 220 m. The fulfillment of pressure head constraint is achieved as the critical node pressure head is greater than the minimum required pressure head constraint (i.e., 30 m). The identification of the critical node is essential as it is very sensible to any variation in the network demands and may cause a violation of the minimum required pressure head constraint.

(d) The optimal diameters of the pipes connected to reservoirs 1 and 2 (i.e. pipes 1 and 9, respectively) are the main parameters that determine the flow rates from the two reservoirs. Large diameters allow high flow rates to pass through, while small diameters allow low flow rates to pass through due to: (i) the flow rate through the pipe connected to the reservoir with a head H_{res} is $Q = 0.25\pi D^2 V = 0.25\pi D^2(2g)^{0.5}(H_{res} - h_f - Z_o - H_o)^{0.5}$ where Z_o and H_o are the elevation and nodal head of node o at the end of pipe, respectively, and (ii) the head loss, h_f, according to the Hazen-Williams friction formula, Eq. (3), is inversely related to the pipe diameter.

The analysis of the flow rate values in Table VII reveals that there is water filling in the additional reservoir (reservoir 2) when the water elevation inside it is less than or equal to 190 m, Figure 10. When Z is greater than 190 m, the sharing of reservoirs 1 and 2 to supply the total nodal demands (1120 m^3/h) at different water levels in reservoir 2 is shown in Fig. 10. Generally, there are 5 ranges of water levels in reservoir 2 which have the following characteristics:

(a) At 150 m ≤ Z ≤ 190 m, there is water filling of reservoir 2 due to the very small diameter of pipe 9, i.e. D_9 = 1 inch, which results in high head loss in comparison with pipe 1 whose diameter is 18 inch. The total flow rate from reservoir 1 is the sum of the total nodal demands (i.e., 1120 m^3/h) and the filling rate of reservoir 2 (i.e., 5.08 m^3/h at Z = 150 m decreasing to 0.482 m^3/h at Z = 190 m).

(b) At 190 m < Z ≤ 200 m, the flow rate from reservoir 1 decreases, while the flow rate from reservoir 2 increases. This is because the optimal diameter D_9 is in the range of 22 inches (Z = 190.01 m) to 18 inches (Z = 200 m), Table VII. Consequently, the flow rate increases from 191.63 m^3/h to 919.36 m^3/h, respectively. Therefore, when the water level in reservoir 2 is at 190.01 m or greater, reservoir 2 is supplying water to the water network. The cause of this phenomenon may be interpreted as follows: reservoir 2 is connected to node 7, which has an elevation of 160 m, through pipe 9, Fig. 5, and the allowable minimum head in each node is constrained to be 30 m. Therefore, it is required for reservoir 2 to be higher than 190 m to overcome the losses in pipe 9 and provide the desired head at node 7.

(c) At 200 m < Z < 215 m, the flow rate from reservoir 1 increases, while the flow rate from reservoir 2 decreases, Table VII. In this range, the optimal diameter D_9 decreases with the increase of Z (i.e., from 16 inches; Z = 205 m; to 12 inches; Z = 210 m) which causes a decrease in the flow rate of reservoir 2.

(d) At 215 m ≤ Z < 225 m, the flow rate from reservoir 1 decreases, while the flow rate from reservoir 2 increases. Noting that the water level in reservoir 1 is 210 m and $H_{res2} > H_{res1}$, i.e., the water level in reservoir 2 is sufficient enough to overcome the water level in reservoir 1, therefore, its flow rate becomes superior to the flow rate of reservoir 1 and it is the main source for feeding the water network.

(e) At 225 m ≤ Z ≤ 550 m, the flow rates from the two reservoirs are approximately constant. This range is the stable part of the network operation and with lower pipe cost.

TABLE VII
OPTIMAL DIAMETER, COST, HEAD AND FLOW RATE RESULTS CORRESPONDING TO THE WATER LEVEL IN RESERVOIR 2 - CASE 2

Water level (m)	**Data**		**Node**								**Cost (Units)**
	Pipe	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	
0	Optimal diameter (m)	0.4572	0.2540	0.4064	0.1016	0.4064	0.2540	0.2540	0.0254	0.0254	419,200
	Head (m)		53.22	30.43	43.39	33.77	30.36	30.35			
	Flow rate (m^3/h)	1122.445	336.910	685.534	32.537	532.997	−202.997	236.910	0.552	−2.445	
185	Optimal diameter (m)	0.4572	0.2540	0.4064	0.1016	0.4064	0.2540	0.2540	0.0254	0.0254	419,200
	Head (m)		53.23	30.44	43.41	33.78	30.38	30.41			
	Flow rate (m^3/h)	1121.722	336.899	684.823	32.547	532.277	−202.277	236.899	0.554	−1.722	
190	Optimal diameter (m)	0.4572	0.2540	0.4064	0.1016	0.4064	0.2540	0.2540	0.0254	0.0254	419,200
	Head (m)		53.24	30.46	43.44	33.80	30.43	30.51			
	Flow rate (m^3/h)	1120.482	336.879	683.603	32.563	531.040	−201.040	236.879	0.558	−0.482	
190.0	Optimal diameter (m)	0.4572	0.2540	0.3556	0.0254	0.3048	0.0762	0.2540	0.0254	0.5588	346,000
	Head (m)		55.23	30.14	45.81	31.75	30.49	30.00			
	Flow rate (m^3/h)	928.372	368.396	459.976	0.980	338.996	−8.996	268.396	0.624	191.628	
190.0	Optimal diameter (m)	0.4064	0.2032	0.3556	0.0508	0.3048	0.0254	0.0254	0.2540	0.5588	294,000
	Head (m)		55.53	41.46	46.17	31.92	31.09	30.00			
	Flow rate (m^3/h)	657.621	100.993	456.628	6.099	330.529	−0.529	0.993	262.908	462.379	
190.1	Optimal diameter (m)	0.4064	0.2032	0.3556	0.0254	0.3048	0.0254	0.0254	0.2540	0.5080	278,000
	Head (m)		55.59	41.52	46.32	31.65	31.24	30.02			
	Flow rate (m^3/h)	652.535	101.003	451.533	0.997	330.536	−0.536	1.003	268.000	467.465	
192	Optimal diameter (m)	0.4064	0.2032	0.3556	0.0254	0.3048	0.0254	0.0254	0.2540	0.3048	266,000
	Head (m)		55.59	41.52	46.32	32.66	31.25	31.03			
	Flow rate (m^3/h)	652.431	100.975	451.456	0.969	330.487	−0.487	0.975	268.056	467.569	
197	Optimal diameter (m)	0.2540	0.2032	0.2032	0.0254	0.0254	0.4064	0.0254	0.2540	0.4064	215,000
	Head (m)		48.33	34.29	37.72	37.93	30.11	36.36			
	Flow rate (m^3/h)	320.687	100.542	120.145	0.465	−0.320	330.320	0.542	268.993	799.313	
200	Optimal diameter (m)	0.2032	0.2032	0.0254	0.0254	0.2032	0.3556	0.0254	0.2540	0.4572	180,000
	Head (m)		45.49	31.48	34.76	41.04	30.30	39.53			
	Flow rate (m^3/h)	200.640	100.128	0.512	−0.228	−119.260	449.260	0.128	270.100	919.360	
205	Optimal diameter (m)	0.2032	0.2032	0.0254	0.0508	0.2032	0.3048	0.0508	0.2032	0.4064	163,000
	Head (m)		44.92	30.60	34.15	30.43	30.05	44.18			
	Flow rate (m^3/h)	204.834	104.320	0.514	3.977	−123.463	453.463	4.320	261.703	915.166	
210	Optimal diameter (m)	0.2540	0.2032	0.2032	0.0254	0.0254	0.2540	0.0254	0.2032	0.3048	144,000
	Head (m)		48.30	34.25	37.67	32.49	30.07	47.40			
	Flow rate (m^3/h)	321.133	100.755	120.378	0.698	−0.321	330.321	0.755	268.546	798.867	
215	Optimal diameter (m)	0.2540	0.2032	0.2032	0.0254	0.0254	0.2540	0.0254	0.2032	0.2540	142,200
	Head (m)		48.31	34.26	37.69	33.75	31.34	48.67			
	Flow rate (m^3/h)	320.961	100.711	120.250	0.651	−0.402	330.402	0.711	268.638	799.039	
220	Optimal diameter (m)	0.2032	0.2032	0.0254	0.0254	0.2032	0.2540	0.0254	0.2032	0.3556	136,000
	Head (m)		45.55	31.57	36.08	43.15	31.62	58.40			
	Flow rate (m^3/h)	200.192	99.744	0.448	−0.296	−119.256	449.256	−0.256	270.552	919.808	
225	Optimal diameter (m)	0.1524	0.0254	0.0254	0.0762	0.1524	0.2540	0.1524	0.2540	0.3556	130,000
	Head (m)		43.49	30.36	31.28	56.48	37.88	63.07			
	Flow rate (m^3/h)	100.949	0.369	0.580	−18.186	−101.234	431.234	−99.631	387.817	1019.051	
250	Optimal diameter (m)	0.1524	0.0254	0.0254	0.1524	0.0254	0.2032	0.1524	0.2540	0.2540	112,200
	Head (m)		43.83	38.16	37.13	64.52	64.52	80.05			
	Flow rate (m^3/h)	99.825	−0.441	0.266	−118.995	−0.739	330.739	−100.441	489.436	1020.175	
300	Optimal diameter (m)	0.1524	0.0254	0.0254	0.0254	0.1524	0.2032	0.1524	0.2032	0.2540	103,200
	Head (m)		44.36	67.67	48.01	94.29	60.46	130.02			
	Flow rate (m^3/h)	98.035	−1.325	−0.640	−1.488	−119.152	449.152	−101.325	372.813	1021.965	
350	Optimal diameter (m)	0.1524	0.0254	0.0254	0.1524	0.0254	0.1524	0.1524	0.2032	0.3048	98,000
	Head (m)		44.79	91.83	89.70	118.59	34.09	185.88			
	Flow rate (m^3/h)	96.580	−1.772	−1.649	−123.219	1.570	328.430	−101.772	494.991	1023.420	
400	Optimal diameter (m)	0.1524	0.0254	0.0254	0.1016	0.0508	0.1524	0.1016	0.2032	0.3048	91,000
	Head (m)		43.81	44.53	31.14	173.01	73.95	235.91			
	Flow rate (m^3/h)	99.881	−0.719	0.600	−108.881	−10.520	340.520	−100.719	479.599	1020.119	
450	Optimal diameter (m)	0.1524	0.0254	0.0254	0.1016	0.0254	0.1524	0.1016	0.2032	0.2032	85,300
	Head (m)		44.00	65.31	32.90	195.02	105.58	260.47			
	Flow rate (m^3/h)	99.248	−1.282	0.530	−117.305	−2.165	332.165	−101.282	488.587	1020.752	
500	Optimal diameter (m)	0.1524	0.0254	0.0254	0.1016	0.0254	0.1524	0.1016	0.2032	0.2032	85,300
	Head (m)		44.81	112.61	77.31	244.10	155.37	310.32			
	Flow rate (m^3/h)	96.492	−2.095	−1.413	−119.173	−2.240	332.240	−102.095	491.268	1023.508	
550	Optimal diameter (m)	0.1524	0.0254	0.0254	0.0254	0.1524	0.1524	0.1016	0.1524	0.2032	83,300
	Head (m)		44.52	55.07	80.88	184.21	93.32	360.38			
	Flow rate (m^3/h)	97.489	−1.021	−1.490	−2.377	−119.112	449.112	−101.021	373.398	1022.511	

The negative sign in flow rates means the contrary of supposed flow direction, Table III.

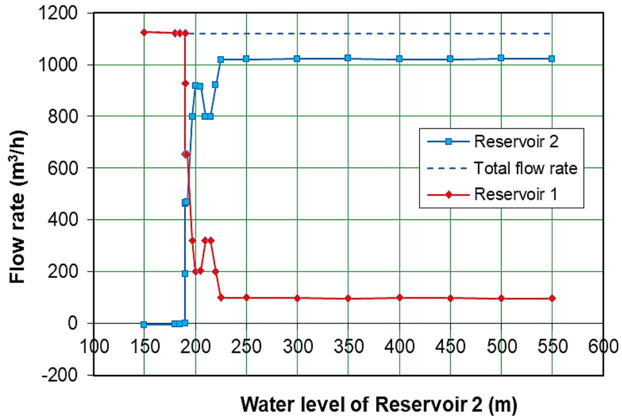


Fig. 10. Variation of the flow rates of Reservoirs 1 and 2 with the water level in reservoir 2, Case 2

On the other hand, the analysis of the network optimal cost in Table VII and Figure 11 reveals that many combinations of pipe sets can give the same optimal cost due to equal lengths of pipes or similar sets of optimal diameters for two water levels of reservoir 2 (e.g., $Z = 450$ m and $Z = 500$ m). In addition, it is noticed that when the water elevation in reservoir 2 becomes higher than 190 m even by 0.01 m (i.e., $Z = 190.01$ m), it starts to work as a water source—instead of filling reservoir 2—with a saving of 17.4% in the optimal piping cost when compared with Case 1 without reservoir 2. Hence, increasing the height of the water level in reservoir 2 above 190 m reduces the optimal piping cost, Fig. 11.

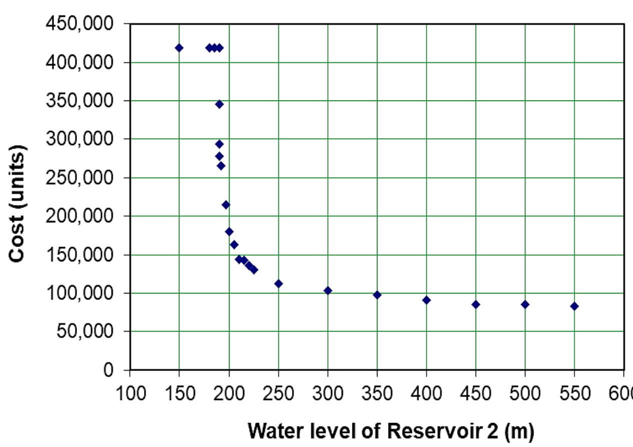


Fig. 11. Variation of the optimal cost with the water level height in reservoir 2, Case 2

The study of the fulfillment of the minimum nodal pressure head constraint (30 m) for some of the studied water levels of reservoir 2 is illustrated in Fig. 12. For each specified water level of reservoir 2, the optimized network has a minimum nodal pressure head higher than 30 m. In addition, the critical node, the node at which this minimum pressure head exists, is given. According to the results of Fig. 12, most of the critical nodes are nodes 6 and 7 for $Z \leq 210$ m where the values of the minimum nodal pressure head are within a maximum of 1 m over the permitted minimum nodal pressure head. This is interpreted by knowing that when the water level of reservoir 2 is 210 m, it becomes equal to that of reservoir 1. When $Z > 210$ m, the pressure head of the critical node is greater than 30 m.

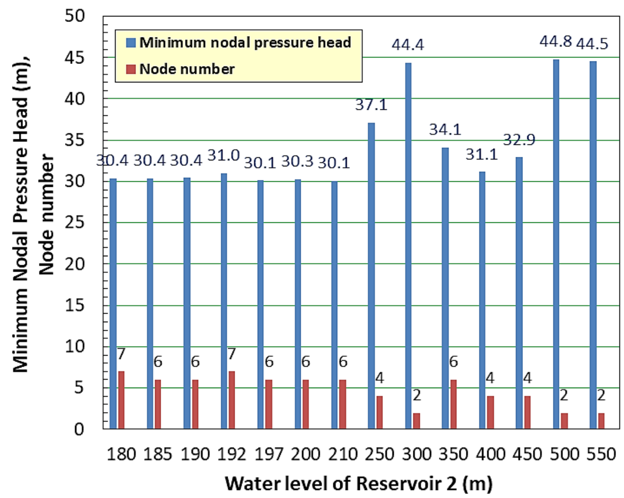


Fig. 12. Minimum nodal pressure head and node number with the water level height in reservoir 2, Case 2

3. *Network Optimization of Case 3*

The third water network includes a reservoir to drive the flow. This three-loop network consists of 11 pipes. Since there are 10 available discrete diameters for each pipeline, the search space consists of 10^{11} elements. The optimal cost obtained from the application of the genetic algorithm is $2,610,500, Table VIII, which is the same value obtained by Costa et al. [33]. The optimal pipe diameters, the heads at the nodes, and the flow rates in the pipes are presented in Table VIII. In addition, the results of the nodal pressure heads and flow rates obtained by both the EPANET code—using the optimal diameters found by the GAPUMPS code—and the GAPUMPS code for Case 3 are shown in the same table. Both results are generally in good agreement. All the nodal heads are greater than 30 m which is the constraint used by our optimization procedure.

The cost evolution resulting from the GAPUMPS code is shown in Figure 13. The optimal solution ($2,610,500) is found after performing 837 hydraulic simulations for a total number of evaluations of 2400. From this figure, it can be observed the rapid convergence tendency to the optimal solution. For the same network, the simulated annealing was applied by Costa et al. [33] and an average number of simulations of 13,454 was employed to find the same optimal cost. Consequently, the micro-genetic algorithm reveals a superior performance for the studied case. The GAPUMPS code takes only 1.15 seconds of CPU time for a total number of evaluations of 2400.

TABLE VIII

DATA, NODAL HEADS AND FLOW RATES FOR THE SIMULATION OF CASES 3 AND 4

			Node									
Data	Pipe	1	2	3	4	5	6	7	8	9	10	11
	Length (m)	2500	2500	2500	2500	2500	2500	2500	2500	2500	2500	2500
	Elevation (m)		200	190	175	180	180	185	185	190		
	Demand (m^3/h)		200	100	100	150	150	100	90	110		
Case 3	Optimal diameter, m (in.)	0.6096 (24)	0.2540 (10)	0.2540 (10)	0.4572 (18)	0.1524 (6)	0.1524 (6)	0.4064 (16)	0.2032 (8)	0.2540 (10)	0.2540 (10)	0.1524 (6)
	Head (m) - EPANET		31.63	30.36	41.1	46.18	42.75	30.11	37.13	30.0		
	Head (m) - GAPUMPS		31.64	30.38	41.12	46.20	42.77	30.12	37.16	30.02		
	Flow rate (m^3/h) - EPANET	1000.0	191.86	113.5	608.14	−21.64	13.5	347.74	86.5	110.4	89.6	20.4
	Flow rate (m^3/h) - GAPUMPS	1000.0	191.841	113.480	608.159	−21.639	13.480	347.793	86.520	110.366	89.634	20.366
Case 4	Optimal diameter, m (in.)	0.6096 (24)	0.2540 (10)	0.1524 (6)	0.4572 (18)	0.1524 (6)	0.1524 (6)	0.3556 (14)	0.2540 (10)	0.2540 (10)	0.2540 (10)	0.1524 (6)
	Head (m) - EPANET		35.1	35.01	34.17	49.47	42.82	31.55	39.79	30.56		
	Head (m) - GAPUMPS		35.11	35.03	34.18	49.49	42.85	31.58	39.81	30.59		
	Flow rate (m^3/h) - EPANET	1000.0	180.82	60.17	619.18	20.66	−39.83	349.71	139.83	119.47	80.53	29.47
	Flow rate (m^3/h) - GAPUMPS	1000.0	180.751	60.156	619.249	20.595	−39.844	349.823	139.844	119.426	80.574	29.426

The negative sign in flow rates means the contrary of supposed flow direction, Table IV.

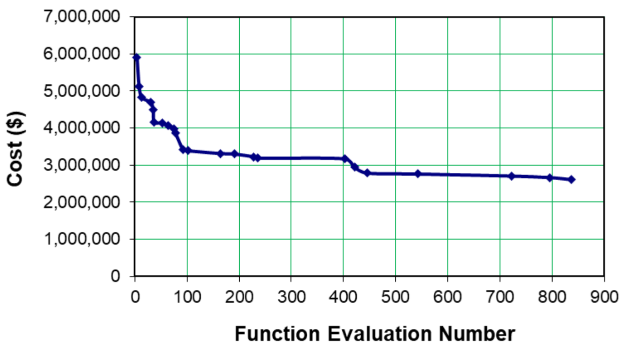


Fig. 13. Cost evolution resulting from the GAPUMPS code for Case 3

4. Network Optimization of Case 4

The fourth network is similar to Case 3 and has a search space of 10^{11} elements. The pump operating point has a discharge of 1000 m^3/h depending on the nodal demands. For candidate pumps number 1, 2 and 3, the pumping pressure is 0, 35.78, and 40.25 m, respectively. The operating heads of pumps 4 to 10 are 48.475, 52.778, 60.458, 64.998, 69.0, 75.00, and 79.994 m, respectively. These values are similar to those given by Geem [35]. It is worth noting that candidate pumps number 2 and 3 do not achieve the minimum required nodal heads , i.e. 30 m, even if the largest available pipe diameters are used. This is due to the hydraulic losses in the pipes. For example, head at node 1 is 24.658 m for candidate pump 2, and 29.13 m for candidate pump 3.

Table VIII shows the simulation results for Case 4 using both the EPANET and GAPUMPS codes and applying the optimal diameters obtained by the GAPUMPS code. Both simulation results are in good agreement, confirming that GAPUMPS is a reliable code in both the simulation and optimization of water distribution networks.

The GAPUMPS code finds the optimal cost at $5,505,050.63, Table IX, the same value obtained by Costa et al. [33] and Geem [35]. Table IX shows the optimal pipe diameters, the heads at the nodes, and the flow rates in the pipes for pump candidates 4 to 10. The fulfillment of the nodal heads (> 30 m) is achieved for the studied pump candidates, but at a higher optimal cost. Because the nodal pressure heads are functions of pipe diameters and pump sizes, some pump sizes (pumps number 2 and 3) in Table IX do not fulfill the required nodal pressure heads.

Table X and Figure 14 show the life cycle costs, namely pipes and pump capital costs, and pump operating costs, used in the objective function. These costs correspond to the optimal solution obtained for each pump candidate from 4 to 10. It is clear that the pipe cost decreases with the candidate pump, while the pump capital cost and the pump operating cost increase with the candidate pump. Although the optimal pipe cost of pump candidate 4 is higher than the other candidates, its total optimal cost is lower. This is traced to the high differences in pump operating cost in comparison to the pipe cost and pump capital cost.

Figure 15 shows the cost evolution resulting from the GAPUMPS code for candidate pump number 4. The code found the optimal solution ($\$5.505 \times 10^6$) after 586 hydraulic simulations for a total number of evaluations of 2400. The figure demonstrates that the micro-genetic algorithm has a rapid convergence tendency in early function evaluation number. For the same network, the harmony search algorithm found the same optimal cost after 2911 hydraulic analyses [35]. Therefore, the micro-genetic algorithm has superiority for the studied case. The run time of the GAPUMPS code for this case was 1.44 seconds for a total number of evaluations of 2400.

TABLE IX
DATA, NODAL HEADS AND FLOW RATES FOR THE OPTIMIZATION OF CASE 4

Pump No.	Data		Node										Optimal Cost ($)
	Pipe	1	2	3	4	5	6	7	8	9	10	11	
1		Without Pump											
2		Fulfillment of minimum required nodal heads (30 m) fails											
3		Fulfillment of minimum required nodal heads (30 m) fails											
4	Optimal diameter (m)	0.6096	0.2540	0.1524	0.4572	0.1524	0.1524	0.3556	0.2540	0.2540	0.2540	0.1524	5,505,050.63
	Head (m)		35.11	35.03	34.18	49.49	42.85	31.58	39.81	30.59			
	Flow rate (m^3/h)	1000.0	180.751	60.156	619.249	20.595	−39.844	349.823	139.844	119.426	80.574	29.426	
5	Optimal diameter (m)	0.5080	0.2540	0.1524	0.4572	0.1524	0.1524	0.3556	0.2540	0.2540	0.2540	0.1524	5,552,165.69
	Head (m)		34.61	34.53	33.68	48.99	42.35	31.07	39.31	30.09			
	Flow rate (m^3/h)	1000.0	180.751	60.156	619.249	20.595	−39.844	349.823	139.844	119.426	80.574	29.426	
6	Optimal diameter (m)	0.5080	0.2540	0.2032	0.4064	0.1524	0.1524	0.3556	0.2032	0.1524	0.2540	0.1524	5,753,147.16
	Head (m)		42.29	37.72	42.00	53.47	46.22	31.91	31.43	30.14			
	Flow rate (m^3/h)	1000.0	220.598	103.792	579.402	16.806	3.792	366.849	96.208	62.556	137.444	−27.444	
7	Optimal diameter (m)	0.5080	0.2540	0.2032	0.4064	0.1524	0.1524	0.2540	0.2032	0.2540	0.1524	0.2032	5,872,606.63
	Head (m)		46.83	38.97	41.54	58.72	43.51	30.69	43.92	31.04			
	Flow rate (m^3/h)	1000.0	246.252	112.441	553.748	33.811	12.441	225.775	87.559	177.973	22.027	87.973	
8	Optimal diameter (m)	0.4572	0.2540	0.2032	0.4064	0.1524	0.1524	0.2540	0.2032	0.2540	0.1524	0.2540	6,009,123.35
	Head (m)		45.36	37.94	40.77	57.16	43.10	30.07	41.02	32.60			
	Flow rate (m^3/h)	1000.0	242.893	111.193	557.107	31.701	11.193	216.357	88.807	190.749	9.251	100.749	
9	Optimal diameter (m)	0.5080	0.2540	0.2540	0.3556	0.1524	0.1524	0.2540	0.1524	0.2032	0.2032	0.1524	6,190,377.74
	Head (m)		56.83	43.22	51.14	63.31	46.74	30.18	42.70	31.18			
	Flow rate (m^3/h)	1000.0	286.354	149.277	513.646	37.077	49.277	236.432	50.723	127.214	72.786	37.214	
10	Optimal diameter (m)	0.4572	0.3048	0.2032	0.3556	0.1524	0.1524	0.2540	0.1524	0.2540	0.1524	0.1524	6,342,736.91
	Head (m)		56.35	55.73	51.18	63.51	51.45	32.42	51.33	30.36			
	Flow rate (m^3/h)	1000.0	300.506	143.668	499.494	56.838	43.668	199.091	56.332	150.403	49.597	60.403	

The negative sign in flow rates means the contrary of supposed flow direction, Table IV.

TABLE X
OPTIMAL PIPE COST, PUMP CAPITAL COST, AND PUMP OPERATING COST OF CASE 4

Candidate Pump No.	Pipe Cost ($)	Pump Capital Cost ($)	Pump Operating Cost ($)	Total Cost of Network ($)
4	2,509,000	1,410,533.05	1,585,517.58	5,505,050.63
5	2,370,500	1,455,414.15	1,726,251.54	5,552,165.69
6	2,228,500	1,547,177.59	1,977,469.57	5,753,147.16
7	2,157,000	1,589,639.70	2,125,966.94	5,872,606.63
8	2,125,000	1,627,274.36	2,256,848.99	6,009,123.35
9	2,055,500	1,681,781.17	2,453,096.57	6,190,377.74
10	1,999,000	1,727,301.95	2,616,434.95	6,342,736.91

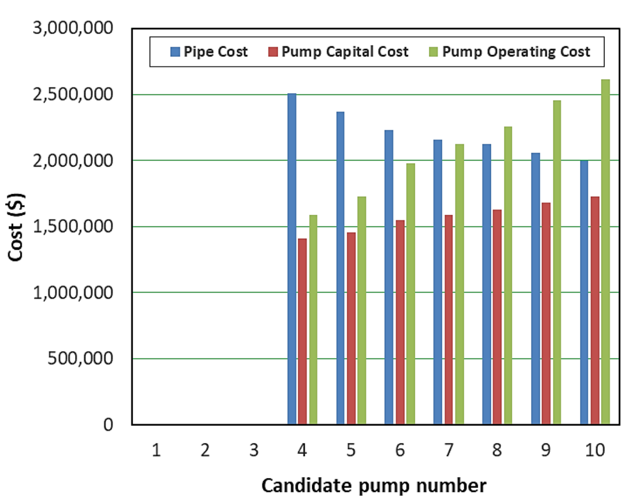


Fig. 14. Optimal pipe cost, pump capital cost, and pump operating cost of Case 4

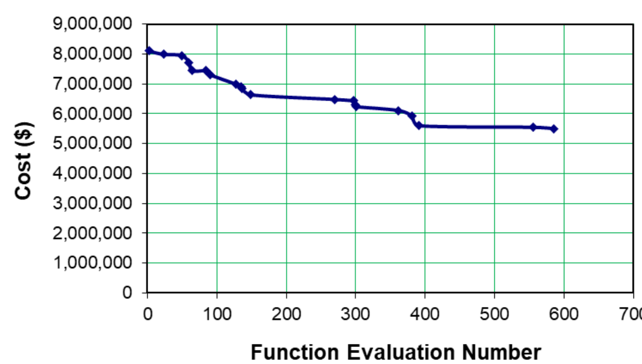


Fig. 15. Cost evolution resulting from the GAPUMPS code for Case 4

The simple evaluation of the centrifugal pump power, Eq. (25), is used in Case 4. Table XI shows the three parameters of Eq. (25), i.e. the maximum discharge, $Q_{\max}$, maximum head, $H_{\max}$, and maximum efficiency, $\eta_{\max}$, for candidate pump numbers 4 to 10 obtained using Eqs. (19), (16), and (22), respectively. Table XII compares the pump power calculated using Eq. (28) with that of Eq. (25). Figure 16 compares the power calculated using Eq. (25) with Eq. (28) proposed by Costa et al. [33]. The maximum discrepancy between the two calculated pump powers is 14.7% for pump candidate number 6. This discrepancy could be due to:

(1) Contrary to the study of Costa et al. [33], which assumed that all pumps have the same efficiency curve which does not depend on the pump curve, the coefficients of the

efficiency curve depend on the maximum flow rate of the pump, Q_{max}, in Eq. (25).

(2) The efficiency curve used by Costa et al. [33], given by Eq. (28), gives a nonzero value to the efficiency at both zero flow rate and maximum flow rate, which is not correct. On the other hand, the efficiency is set to zero at zero flow rate in the derivation of Eq. (25).

TABLE XI
PARAMETERS OF EQ. (25) FOR EACH PUMP OF CASE 4

Candidate Pump No.	Q_{max} (m^3/s)	H_{max} (m)	η_{max} (%)
4	0.7087	55.1	65.76
5	0.6834	65.0	65.76
6	0.8739	67.0	65.76
7	0.8164	75.0	65.76
8	0.7807	79.0	65.76
9	0.7527	85.0	65.76
10	0.8056	91.5	65.76

TABLE XII
COMPARISON BETWEEN PUMP POWER CALCULATED USING EQ. (28) OF [33] AND EQ. (25), CASE 4

Candidate Pump No.	Power calculated using Eq. (28) [33] (kW)	Power calculated using Eq. (25) (kW)	Discrepancy (%)
4	201.998	210.704	4.3
5	219.928	226.632	3.0
6	251.933	288.860	14.7
7	270.852	299.965	10.7
8	287.526	311.865	8.5
9	312.529	333.666	6.8
10	333.338	366.817	10.0

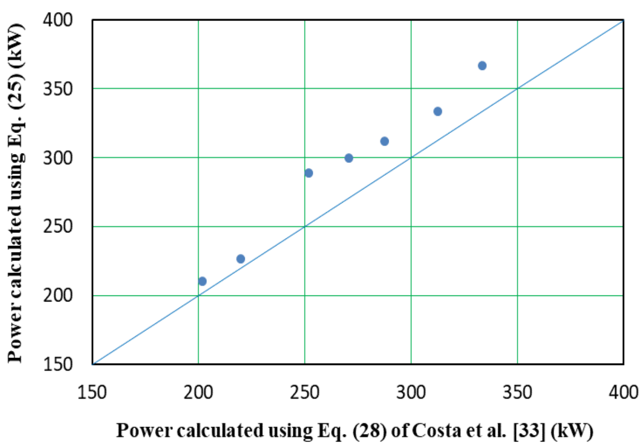


Fig. 16. Comparison of the power calculated using Eq. (25) and using Eq. (28) [33]

VII. CONCLUSIONS

In this paper, both hydraulic simulations and optimization strategies are used in the design and operation of water distribution networks. Micro-genetic algorithm for a single objective function has been coupled with the Newton-Raphson method for both the hydraulic simulation and design of water distribution networks.

The developed code, GAPUMPS, is verified through four water distribution networks for assessing minimum cost; three cases are for optimization of networks without pumps, whereas the fourth case is for optimization of a network with a centrifugal pump. The following conclusions can be asserted:

(a) The hydraulic simulations of the four cases using EPANET and GAPUMPS codes give almost identical nodal heads and flow rates revealing the reliability of the proposed GAPUMPS code.

(b) The optimization of Case 1 for the two-loop network with a single reservoir gives the cost of 419,000 units previously found by many researchers but with a lower number of function evaluations.

(c) The optimization of Case 2 for the two-loop network with two reservoirs is devoted to study the effect of the water level in reservoir 2 (180 m to 550 m) on the network optimization. There is water filling in reservoir 2 if the water level is less than or equal to 190 m, and beyond that, reservoir 2 supplies the total nodal demands. Similarly, the optimal cost reduces with the increase of the water level in reservoir 2 with a saving of 17.4% in the optimal piping cost for a level of 190.05 m in comparison with Case 1 without reservoir 2.

(d) The optimization of Case 3 for the three-loop network without pump results in an optimal solution of $2,610,500 after performing 837 hydraulic simulations. On the other hand, Costa et al. [33] found the same optimal cost with an average number of simulations of 13,454 using the simulated annealing optimization. Consequently, the GAPUMPS code reveals a superior performance for the studied case.

(e) The optimization of Case 4 for the three-loop network with a pump finds the optimal cost at $5,505,050.63, similar to that obtained by Costa et al. [33] and Geem [35]. However, the function evaluation number using the GAPUMPS code is 586, whereas it is 2911 hydraulic analyses using the harmony search algorithm [35].

(f) The comparison of the pump power equation, Eq. (25), and the GAPUMPS code for Case 4 using Eq. (28) [33] reveals that the maximum discrepancy is 14.7% for pump candidate number 6. The discrepancy is attributed to a single efficiency curve for all pumps and a nonzero efficiency at zero flow rate [33].

Therefore, the comparison of the performance of micro-genetic algorithm with those obtained by other optimization techniques shows that the micro-genetic algorithm yields better performance in terms of optimal network design cost and/or computational speed, i.e. algorithm convergence with fewer iterations, therefore saving time.

AUTHORS CONTRIBUTION

The following summarizes author statement outlining their individual contributions to the paper using the relevant roles:

1. *George Ghali*: Developing the computer code, Data collection and tools, data analysis and interpretation,

methodology, and drafting the article. In addition, the corresponding author is responsible for ensuring that the descriptions are accurate and agreed by all authors.
2. *Berge Djebedjian*: Conception and design of work, Developing the computer code, data interpretation, supervision, and critical revision of the article.
3. *Abdel-Rahim Dohina*: Conception and design of work, data interpretation, supervision, and critical revision of the article.
4. *Yahia M. Fouda*: Conception and design of work, supervision, methodology, and final approval of the version to be published.

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ARABIC TITLE:

أمْثَلة تكلفة شبكات توزيع المياه بدون صهاريج تخزين

ARABIC ABSTRACT:

تبحث هذه الورقة البحثية في التصميم الأمْثَل لشبكات توزيع المياه بدون سعة تخزينية باستخدام الأمْثَلة الرياضية. يتم استخدام الخوارزمية الجينية الدقيقة والمحاكاة الهيدروليكية لشبكة المياه للحصول على الأقطار المُثْلى لأنابيب الشبكة. دالة الهدف الواحد هي التكلفة الرأسمالية للأنابيب والمضخة وتكلفة تشغيل المضخة في ظل الوفاء بمتطلبات الضغط الأدنى عند العقد. تم تصميم منهجية الأمْثَلة (GAPUMPS - الخوارزمية الجينية للمضخات) لتقليل استهلاك الطاقة. يتم توضيح الإجراء الموصوف في أربع دراسات حالة لشبكات توزيع المياه مع أو بدون مضخات. ثلاث دراسات حالة هي بالفعل شبكات معيارية تقليدية في المؤلفات : الشبكة ذات الحلقتين والشبكة ثلاثية الحلقات. أظهرت النتائج توافق جيد مع الدراسات السابقة المتعلقة بدراسات الحالة الأربع. حصل استخدام الخوارزمية الجينية الدقيقة على تكاليف مُثْلى سابقة أفضل أو مماثلة من حيث عدد تقييمات الدالة بالمقارنة مع تلك الخاصة بالخوارزميات التطورية الأخرى.