OPTIMAL ANALYSIS AND DESIGN OF WATER DISTRIBUTION SYSTEMS USING TUG OF WAR OPTIMIZATION ALGORITHM

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ABSTRACT

In this study, the recently developed method, Tug of War Optimization (TWO), is employed for simultaneous analysis, design and optimization of Water Distribution Systems (WDSs). In this method, analysis procedure is carried out using Tug of War Optimization algorithm. Design and cost optimization of WDSs are performed simultaneous with analysis process using an objective function in order to satisfying the analysis criteria, design constraints and cost optimization. A number of practical examples of WDSs are selected to demonstrate the efficiency of the presented algorithm. The findings of this study clearly signify the efficiency of the TWO algorithm in reducing the water distribution networks construction cost.

Keywords: analysis; design; optimization; water distribution system; tug of war optimization.

1. INTRODUCTION

The main goal of water distribution network optimization is to minimize the costs while satisfying the performance and hydraulic constraints required by the design codes and specifications. This involves determining the commercial diameter for each pipe in the network while satisfying the minimum head pressure at each node. It is essential to investigate and establish a reliable network ensuring adequate head. However, the optimal network design is quite complicated due to nonlinear relationship between flow and head loss and the presence of discrete variables, such as market pipe sizes [1]. In addition, the objective function, which represents the cost of the network, is also nonlinear and causes

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great difficulty in the design optimization of the network. Researchers in recent years have focused on probabilistic approach to overcome these difficulties considering a combination of random and deterministic steps [2, 3]. Genetic Algorithms (GA) [1], GLOBE and Shuffled Frog Leaping Algorithms (SFLA) [3], Harmony search (HS) [4], are the few widely used algorithms in this field of study.

One of the most imperative fields in which the optimization and resource management needs special consideration is water distribution system. Water distribution system, a hydraulic infrastructure consisting of elements such as pipes, tanks, reservoirs, pumps, and valves etc., is crucial to provide water to the consumers. This configuration is usually simplified by the graph layout that has a number of nodes denoting the places in urban area, line denoting the pipes, and other features such as reservoir and pumps. The construction and maintains of water distribution system pipelines to supply water can cost millions of dollars every year. Therefore, it is essential to investigate and establish a reliable network ensuring adequate head. However, the optimal network design is very complicated due to nonlinear relationship between flow and head loss and the presence of discrete variables.

Due to the high costs associated with the construction of water distribution systems (WDSs) much research has been dedicated to the development of methods to minimize the capital costs associated with such infrastructure.

Traditionally water distribution system design is based on trial-and-error methods employing the experience. However, in the light of the optimization of cost and profits, designing the best layout of water supply system counting the best selection of water demands and pipe length and diameter within the millions of possible configuration, attracted a large amount of literature during the last decades. The majority of literatures have focused on cost; though, other ones deal with other aspects of designing, such as reliability.

The research in optimization has attracted many researchers focusing on various programming methods such as linear and non-linear programming [5, 6]. Alperovits and Shamir [7] reduced the complexity of an original nonlinear problem by solving a series of linear sub-problems. In this method a linear programming problem is solved for a given flow distribution, and then a search is conducted in the space of the flow variables. This method was followed and other methods were developed, examples of which are Quindry et al. [8], Goulter et al. [9], and Fujiwara and Kang [10] who used a two-phase decomposition method. Meta-heuristic methods such as Genetic Algorithms [11, 12], Ant colony optimization [13, 14], the Shuffled Frog-Leaping Algorithm [3] were also utilized in several optimization approaches for water distribution networks. Geem [4], who developed harmony search (HS) and particle-swarm harmony search (PSHS) and Eusuff and Lansey [3], who proposed an SFLA models are also employed their techniques for water distribution system optimization. Tolson et al. [15] developed a hybrid discrete-dynamically dimensioned search (HD-DDS) algorithm to perform optimal design of water distribution system.

From recent metaheuristic algorithms one can refer to Kaveh and Khayatazad [16] for the Ray Optimization algorithm, Kaveh and Mahdavi [17] for Colliding Bodies Optimization, Kaveh and Ilchi Ghazaan [18] Enhance Colliding Bodies Optimization algorithms, and Kaveh and Zolghadr [19] for the Tug of War Optimization method (TWO). The TWO algorithm is used in this study as an optimization algorithm and it also performs as an analyzer instead of classic analyzer such as Newton-Raphson approach. In the classic methods pipe demands are often calculated using indirect methods and pre-selected pipe

sizes are utilized. However in this paper, the pipe sizes and demands are considered as the optimization variables leading to simultaneous analysis, design and optimization.

The present paper is organized as follows: In the next section, the WDSs optimization problem is introduced. In Section 3, the optimization algorithm (TWO) is briefly introduced and the new method for analysis and design of WDSs is described. In Section 4, the cost of WDSs as the design objective function is minimized, and finally Section 5 concludes the paper.

2. WATER DISTRIBUTION NETWORK OPTIMIZATION PROBLEM

The water distribution network optimization problem is defined as the selection of the most desirable configuration of circulation network considering the allowable pipe diameter and water demand in each point while satisfying various possible objectives such as network reliability, redundancy, water quality. One of the most common and favorable objective function of water distribution system is considered as minimizing the network arrangement cost, by suitable selection of pipe diameters and lengths. The network cost is calculated as the sum of the pipe costs where pipe costs are expressed in terms of cost per unit length. Total network cost is computed as follows:

$$C = \sum_{i=1}^{N} f(D_i, L_i) \tag{1}$$

where $f(D_i, L_i)$ is the cost of the ith pipe, with diameter D_i and length L_i, and N is the number of pipe in the network configuration.

In each engineering problem two phases should be performed to achieve a goal, analysis and design. In the water distribution systems problem, which is a complex system of pipes, the goal is defined as the length and diameters of the pipes forming a complex configuration while obtaining the required water demands at certain points of the network.

2.1 Analysis phase

In the analysis phase, the goal is to achieve a distribution of water for the postulated configuration of pipe length and diameters among an infinite number of distributions. This is achieved in the light of the fact that only our proposed distribution should satisfy the continuity equation in each node, and satisfy the hydraulic head loss principle in the system loops. In other word, only a few distributions can assure the continuity equation in each node and through these distributions, only one distribution can satisfy the hydraulic head loss equations. Continuity equation or mass conservation at each node is given by

$$\sum Q_{in} - \sum Q_{out} = Q_e \tag{2}$$

where Q_{in} is the volumetric flow rate to the node, Q_{out} is the flow rate out of the node, and Q_e is the external inflow rate to the node.

Considering that each loop is actually a series of pipes of the configuration, where the differences between the head losses of the two end nodes of its pipes should be summed in order to find the head loss of the entire loop. For conservation of energy this sum should be equal to zero. Obviously if a loop has other features such as pumps, its energy interactions should also be added to the conservation equation formula as

$$\sum h_f - \sum E_p = 0 \tag{3}$$

where h_f is the hydraulic head loss calculated by the Hazen-Williams or Darcy-Weisbach formulae and E_p is the energy added to water at the loop by a pump. The above equation is also known as the hydraulic head loss equation.

For the analysis of a water distribution system fundamental principles of water systems are used. The principle of water branching has an interesting analogy with characteristics of electric circuit when rate of the flow corresponds to the electric current and the head loss correspond to the drop in potential. The hydraulic head loss, between two nodes i and j, can be expressed by Hazen-Williams formula as:

$$h_f = \omega \frac{L}{C^\alpha D^\beta} Q^\alpha \tag{4}$$

where ω is a numerical conversion constant; α is a coefficient equal to 1.85; and β is coefficient equal to 4.87.

Based on the analogy between the electric circuits and the pipe branching, when two pipes are in the form of series, the head loss in this series configuration will be equal to the sum of head losses of the constituting pipes (determined by Eq. (4)), and the flow is equal to the flow rate of each pipe.

$$\Delta h_t = \omega \frac{L_a}{C_a^\alpha D_a^\beta} Q^\alpha + \omega \frac{L_b}{C^\alpha D_b^\beta} \tag{5}$$

$$Q_t = Q_a = Q_b \tag{6}$$

where a and b denote the pipe a and pipe b which are used in the series configuration of pipe network.

Now considering the fact that each network may include a combination of parallel and series arrangement of branching pipes, the formulation of water distribution network is obvious. However a network configuration has other features such as loops and reservoir, which should be carefully dealt with, and as a result other equation should be set to achieve the best supply system.

2.1 Design phase

In the design phase of the water distribution system, the pipe diameters satisfying the water demand in each node and place of the urban area should be determined.

As previously mentioned, in this section the third imperative requirement of the water distribution system design should be set. This requirement is the minimum pressure requirement which is usually has a limitation to prevent system failure. Thus during the network configuration assortment, the pressure in each point should be checked. For each node in the network, the minimum pressure constraint is given in the following form:

$$H_j \geq H_j^{\min}; j = 1,...,M \tag{7}$$

where H_j, $H_j^{\min}$ and M denote the pressure head at node j, minimum required pressure at node j, and the number of nodes in the network, respectively. Other requirements such as reliability, minimum and maximum limitation of the velocity and the maximum pressure should be satisfied in the design phase.

To attain the network that satisfies the water requirement, conservation of mass and energy equations in each node and loop should be coupled and solved. These equations can be arranged in the following form:

$$\mathbf{H} \times \mathbf{q}_p - \begin{bmatrix} \mathbf{Q} \\ \mathbf{Null(M,1)} \end{bmatrix} = 0 \tag{8}$$

where Q is the demand in each node, and $\mathbf{Null(M,1)}$ is a $M \times 1$ zero vector with M being the number of loops. This zero vector indicates that in each loop the summation of pipe's head loss should be zero, as the conservation of energy implies. It can be seen that N demands node (N conservation of mass equation for each node) and M loop energy conservation equation, construct the above form of equations. q_p denotes the flow rate of each pipe.

The matrix H consists of two essential parts. The first part corresponds to the equation of the conservation of mass consisting of some positive and negative 1, indicating the input and output flow rate of each node. Besides there are some 0 entries which obviously signify the pipes that are not relevant to considered node in that equation whose flow rate is considered in q matrix in the same row. The second part of H corresponds to M loops containing some positive and negative coefficients which are determined considering the flow rate direct in each pipe, being assumed at the first step of the analysis (conservation of mass) and the postulated direction of the loops. These coefficients are determined using the Hazen-Williams formula. As previously mentioned the primary directions assigned to the pipes may not satisfy the conservation of energy equation, and the correct directions are decided in the process of design. As an example, Fig. 1 shows a fundamental simple WDSs example for which the following equations should be satisfied:

$$\sum_{l \to k} \pm q_l = -Q_K \quad k = 1,2,...,5 \tag{9}$$

$$\sum_{l \to m} \pm A_l |q_l|^{n-1} q_l = 0.0 \quad m = 1,2 \tag{10}$$

$$
\begin{bmatrix}
1 & -1 & -1 & 0 & 0 & 0 \\
0 & 1 & 0 & -1 & -1 & 0 \\
0 & 0 & 1 & 1 & 0 & -1 \\
0 & 0 & 0 & 0 & 1 & 1 \\
0 & A_2|q_2|^{n-1} & -A_3|q_3|^{n-1} & A_4|q_4|^{n-1} & 0 & 0 \\
0 & 0 & 0 & -A_4|q_4|^{n-1} & A_5|q_5|^{n-1} & -A_6|q_6|^{n-1}
\end{bmatrix}
\begin{bmatrix}
q_1 \\ q_2 \\ q_3 \\ q_4 \\ q_5 \\ q_6
\end{bmatrix}
=
\begin{bmatrix}
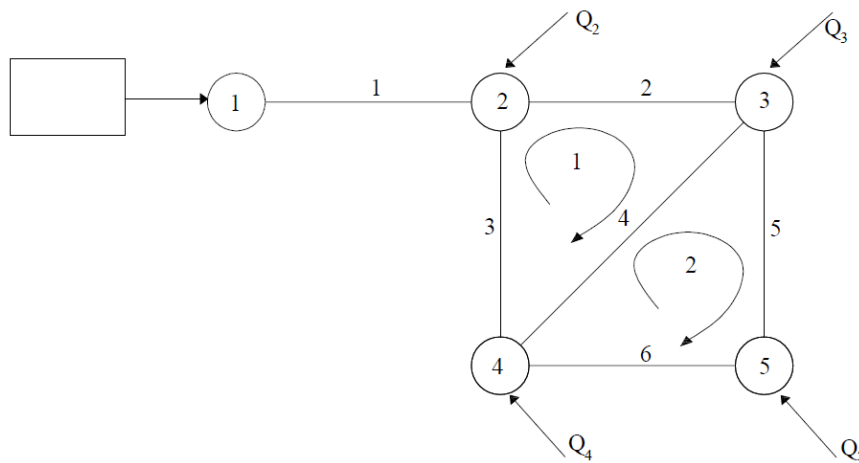-Q_2 \\ -Q_3 \\ -Q_4 \\ -Q_5 \\ 0.0 \\ 0.0
\end{bmatrix}
\quad (11)
$$

Figure 1. An example of simple fundamental WDSs

where $A = \omega \dfrac{L}{C^\alpha D^\beta}$. In the first 4 rows of this H matrix (corresponding to 4 nodes where the water is being used) the first part of H is presented. In the first row of matrix, the entry for the pipe number 1 is positive since the direction of the flow in this pipe has an input role to the point. While the pipes 2 and 3 play the output role. As an illustration of the second part, considering the loop 1, one can say that: the direction of the pipes 2 and 4 are the same as the direction of the loop 1, thus have positive signs. While the pipe number 3 acts in the reverse direction of the loop direction.

Finally it should be mentioned that, in this study, similar to that of the Fujiwara and Kang [10], to achieve a better design, the configuration of series pipes which have the standard pipe diameters are used. For example if the program chooses the pipes with the 38 inch diameter for the system which does corresponds to neither the standard 30 inch nor to the 40 inch pipes, the later subroutine would change the pipe to two series pipes. One of the pipes would have diameter equal to 30 inch and the other will be 40 inch. This exchange should be made such that the sum of the lengths of two pipes is the same as the primary pipe. Since these two pipes should have the same demand as that of the primary pipe, and the total hydraulic head loss of these two pipes should be equal to the primary pipe.

3. THE TUG OF WAR OPTIMIZATION ALGORITHM

In this section, the new meta-heuristic algorithm developed by Kaveh and Zolghadr [19] is introduced. The TWO is a population-based search method, where each agent is considered as a team engaged in a series of tug of war competitions. The weight of the teams is determined based on the quality of the corresponding solutions, and the amount of pulling force that a team can exert on the rope is assumed to be proportional to its weight. Naturally, the opposing team will have to maintain at least the same amount of force in order to sustain its grip of the rope. The lighter team accelerates toward the heavier team and this forms the convergence operator of the TWO. The algorithm improves the quality of the solutions iteratively by maintaining a proper exploration/exploitation balance using the described convergence operator. A summary of this method is described in the following steps.

Step 1*: Initialization*
The initial positions of teams are determined randomly in the search space:

$$x_{ij}^0 = x_{j,min} + rand(x_{j,max} - x_{j,min}) \qquad j = 1,2,...,n \tag{12}$$

where x_{ij}^0 is the initial value of the *j*th variable of the *i*th candidate solution; $x_{j,\max}$ and $x_{j,\min}$ are the maximum and minimum permissible values for the *j*th variable, respectively; *rand* is a random number from a uniform distribution in the interval [0, 1]; *n* is the number of optimization variables.

Step 2*: Evaluation and weight assignment*
The objective function values for the candidate solutions are evaluated and sorted. The best solution so far and its objective function value are saved. Each solution is considered as a team with the following weight:

$$W_i = 0.9(\frac{fit(i) - fit_{worst}}{fit_{best} - fit_{worst}}) + 0.1 \qquad i = 1,2,...,N \tag{13}$$

where *fit(i)* is the fitness value for the *i*th particle; The fitness value can be considered as the penalized objective function value for constrained problems; *fitbest* and *fitworst* are the fitness values for the best and worst candidate solutions of the current iteration; According to Eq. (13) the weights of the teams range between 0.1 and 1.

Step 3*: Competition and displacement*
In TWO each team competes against all the others one at a time to move to its new position. The pulling force exerted by a team is assumed to be equal to its static friction force $(W\mu_s)$. Hence the pulling force between teams *i* and *j* ($F_{p,ij}$) can be determined as the

max$\{ W_i\mu_s, W_j\mu_s \}$. Such a definition keeps the position of the heavier team unaltered.

The resultant force affecting team i due to its interaction with heavier team j in the kth iteration can then be calculated as follows:

$$F_{r,ij}^{k} = F_{p,ij}^{k} - W_i^{k}\mu_k \tag{14}$$

where $F_{p,ij}^{k}$ is the pulling force between teams i and j in the kth iteration and μ_k is coefficient of kinematic friction. Consequently, team i accelerates towards team j:

$$a_{ij}^{k} = (\frac{F_{r,ij}^{k}}{W_i^{k}\mu_k})g_{ij}^{k} \tag{15}$$

where a_{ij}^{k} the acceleration of team i towards team j in the kth iteration; g_{ij}^{k} is the gravitational acceleration constant which is defined as:

$$g_{ij}^{k} = X_j^{k} - X_i^{k} \tag{16}$$

where X_j^{k} and X_i^{k} are the position vectors for candidate solutions j and i in the kth iteration. Finally, the displacement of the team i after competing with team j can be derived as:

$$\Delta X_{ij}^{k} = \frac{1}{2}a_{ij}^{k}\Delta t^2 + \alpha^k(X_{max} - X_{min})\circ(-0.5 + rand(1,n)) \tag{17}$$

The second term of Eq. (17) introduces randomness into the algorithm. This term can be interpreted as the random portion of the search space traveled by team i before it stops after the applied force is removed. Here, α is a constant chosen from the interval [0,1]; X_{max} and X_{min} are the vectors containing the upper and lower bounds of the permissible ranges of the design variables, respectively; $\circ$ denotes element by element multiplication; $rand(1,n)$ is a vector of uniformly distributed random numbers.

It should be noted that when the team j is lighter than the team i, the corresponding displacement of the team i will be equal to zero (i.e. ΔX_{ij}^{k}). Finally, the total displacement of the team i in iteration k is equal to:

$$\Delta X_i^{k} = \sum_{j=1}^{N} \Delta X_{ij}^{k} \tag{18}$$

The new position of the team i at the end of the kth iteration is then calculated as:

$$X_i^{k+1} = X_i^k + \Delta X_i^k \qquad (19)$$

Step 4*: Side constraint handling*

It is possible for the candidate solutions to leave the search space and it is important to deal with such solutions properly. This is especially the case for the solutions corresponding to lighter teams for which the values of ΔX is usually bigger. Different strategies might be used in order to solve this problem. In this study, such candidate solutions are simply brought back to their previous permissible position (Flyback strategy) or they are regenerated randomly.

Step 5*: Termination*

Steps 2 through 5 are repeated until a termination criterion is satisfied.

As explained in Section 2, the matrix H known as the stability matrix of the network cannot be solved by a direct method. Thus this matrix is solved utilizing different indirect approaches such as Newton-Raphson and etc. Classic methods that use the above mentioned indirect approaches perform the analysis and design steps in separate steps requiring a considerable amount of computational time. But in the presented method analysis, design and optimization steps are performed simultaneously. In order to analyze a network we have to find a set of pipe demands that satisfies the Eq. (8) mentioned in Section 2.

In the present approach, the analysis phase is performed using the TWO algorithm by searching a vector of the pipe demands that satisfies the above equation. The left-hand side of this equation is a zero vector and should be changed to a scalar. The best is to find its norm. If this norm is zero, all the entries should be zero. When the norm of a vector equals to zero then all the arrays of the vector equals to zeros. Considering the norm of the above matrix as the analysis constraints can be a reliable fundamental to this goal. Then simultaneous with the design, the analysis phase will be performed by considering the following objective function as the optimization goal function:

$$f(\boldsymbol{q}_p, D) = \sum_{i=1}^{L} l_i \times cost(D_i) \times (1 + norm(\boldsymbol{H} \times \boldsymbol{q}_p = \begin{bmatrix} \boldsymbol{Q} \\ \boldsymbol{Null(M,1)} \end{bmatrix})) + \sum_{i=1}^{L} g_i(q_i, D_i) \qquad (20)$$

Figure 2. shows the schematic procedure of designing and analysis of a water distribution system using the TWO algorithm which is used in this paper.

4. NUMERICAL EXAMPLES

In order to assure that this method is reliable and capable in this field of engineering; three famous networks are selected from literature, which are studied by many other researchers. The following sections explain the comparative study of cost optimization of water distribution system for these networks.
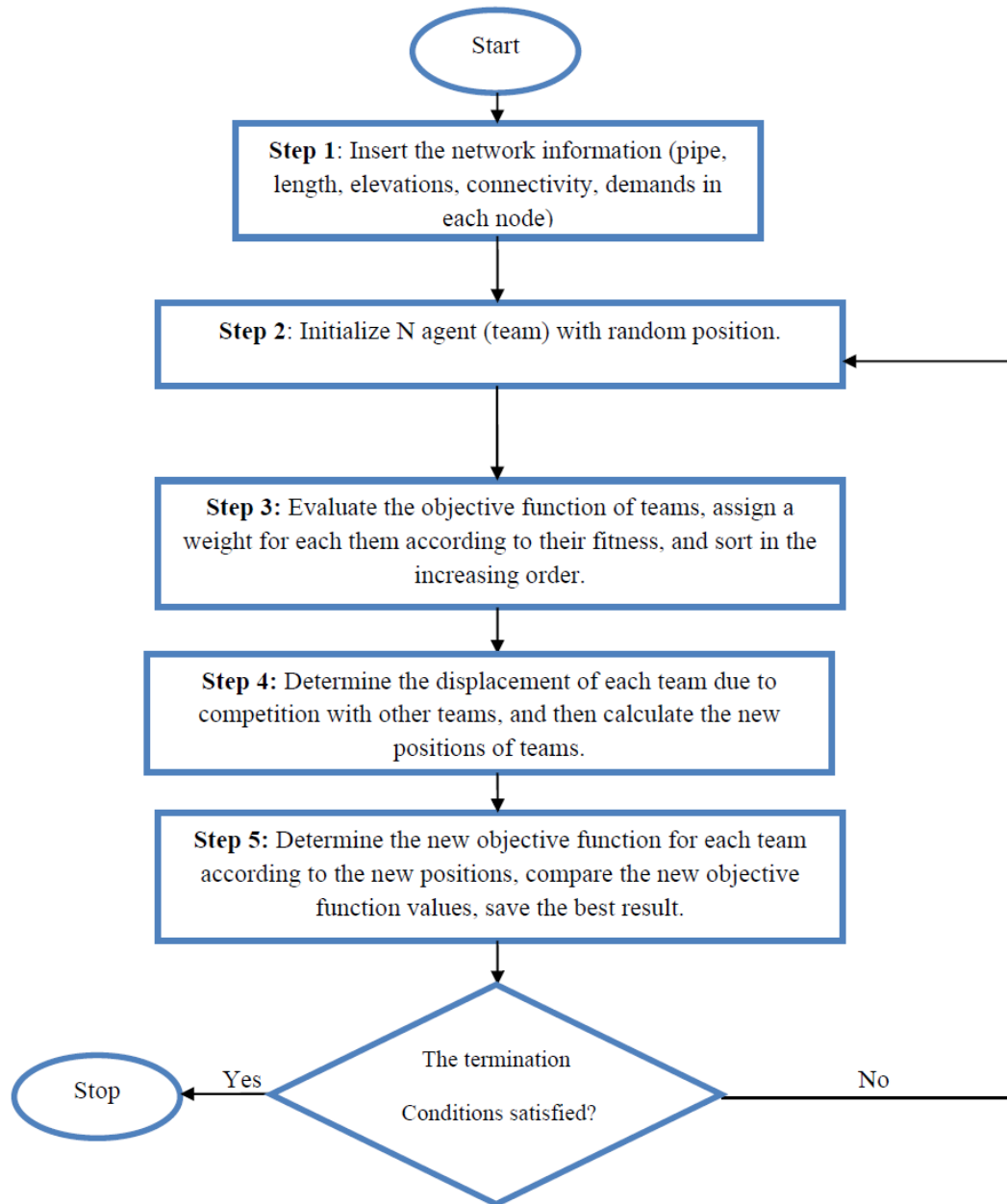
Figure 2. Flowchart of the present study procedure

4.1 A two-loop network

The two loop network, shown in Fig. 3, was first presented by Alperovits and Shamir [7] for implementation of linear programming to acquire the least cost solution, considering the network pipes weight. Later this basic configuration was employed by different authors [5-9] for comparison of their results for optimal design of water distribution system as an illustrative simple network. This network consists of 8 pipes, 7 nodes, and 2 loops. The

network is fed by gravity from a constant reservoir, which has 210 m fixed head. The length of all the pipes is assumed to be 1000m with a Hazen-Williams coefficient (C) being equal to 130. Allowed pipe diameter and corresponding costs are available in Table 1 [10]. The minimum head limitation in each pipe is set to 30m above ground level. Here $\omega = 10.5088$ is employed for the Hazen-Williams formulation as Savic and Walters [2].
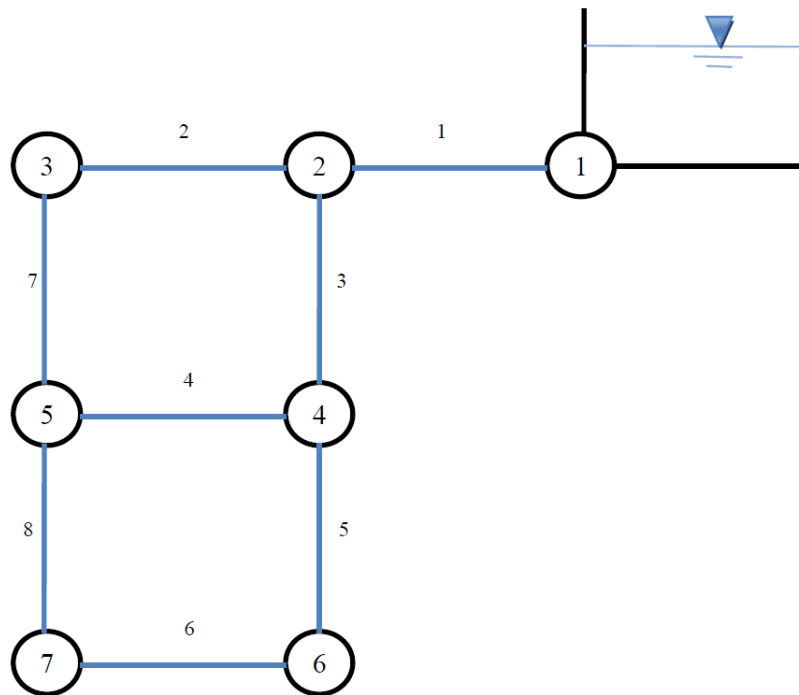

Figure 3. Two-loop water distribution network

Table 2 lists the optimal network solutions, and total network cost obtained using the TWO algorithm. The pressure at each node is shown in Table 3. As can be seen, in all nodes the minimum nodal head requirement is satisfied. Although the least cost is obtained by Kaveh et al. [20], the TWO has a good performance compared to other methods as shown in Table 2.

Table 1: Candidate pipe diameters

Network	Candidate diameter	Corresponding cost
Two-loop	{1, 2, 3, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24} in inches	{2, 5, 8, 11, 16, 23, 32, 50, 60, 90, 130, 170, 300, 550} in dollar/meter
Hanoi	{12, 16, 20, 24, 30, 40} in inches	{45.726,70.4,98.378, 129.333, 180.748, 278.28} in dollar/meter
Go Yang	{80, 100, 125, 150, 200, 250, 300, 350} in millimeters	{37,890; 38,933; 40,563; 42,554; 47,624; 54,125; 62,109; 71,524} in won/meter

Table 2: Comparison of the pipe diameters for the two-loop network

Pipe No.	Alperovits and Shamir	Goulter et al.	Kessler and Shamir	Kaveh et al. [21]		Kaveh et al. [20]		Present work (TWO)	
				Pipe length (m)	Pipe diameter (in)	Pipe length (m)	Pipe diameter (in)	Pipe length (m)	Pipe diameter (in)
1	20 18	20 18	18	L1=595.52 L2=404.48	D1=18 D2=16	L1=987.85 L2=12.15	D1=18 D2=16	L1=996.12 L2=3.88	D1=18 D2=16
2	8 6	10	12 10	602.78 397.22	10 8	74.8 925.2	12 10	283.10 716.90	12 10
3	18	16	16	94.36 905.64	20 18	998.25 1.75	16 14	979.39 20.61	16 14
4	8 6	6 4	3 2	582.75 417.25	8 6	981.93 18.07	3 2	223.99 776.01	3 2
5	16	16 14	16 14	806.91 193.09	16 14	934.62 65.38	16 14	858.98 141.02	16 14
6	12 10	12 10	12 10	174.46 825.54	10 8	996.85 3.15	10 8	213.39 786.61	12 10
7	6	10 8	10 8	934.91 65.09	8 6	751.14 248.86	10 8	769.22 230.78	10 8
8	6 4	2 1	3 2	978.63 21.37	2 1	996.25 3.75	2 1	997.04 2.96	2 1
Cost ($)	497,525	435,015	417,500	432,358		415,070		418,520	

Table 3: Optimal pressure heads for two-loop network

Pipe No.	Min pressure required (m)	Pressure
1	-	-
2	30	53.25
3	30	31.84
4	30	43.55
5	30	30.84
6	30	30.15
7	30	30.81

4.2 Hanoi water distribution network

The Hanoi network is a real network that is formerly studied by Fujiwara and Kang [10] in Vietnam. This network is shown in Fig. 4. This water circulation network can be considered as a medium size network by means of including 32 nodes, 34 pipes, 3 loops and 1 gravity reservoir with a 100m fixed head for its feeding. As the previous example, the Hazen-Williamz coefficient C=130 is employed for network water distribution equations. The tolerable of the pipe diameters, which have pronounced as the difference in upper limitation diameter with the two-loop network, is displayed in Table 1. The water required in this network is much higher than the accustomed demands for other ones so for satisfying these demands, the maximum velocity limitation is set to 7 m/s.
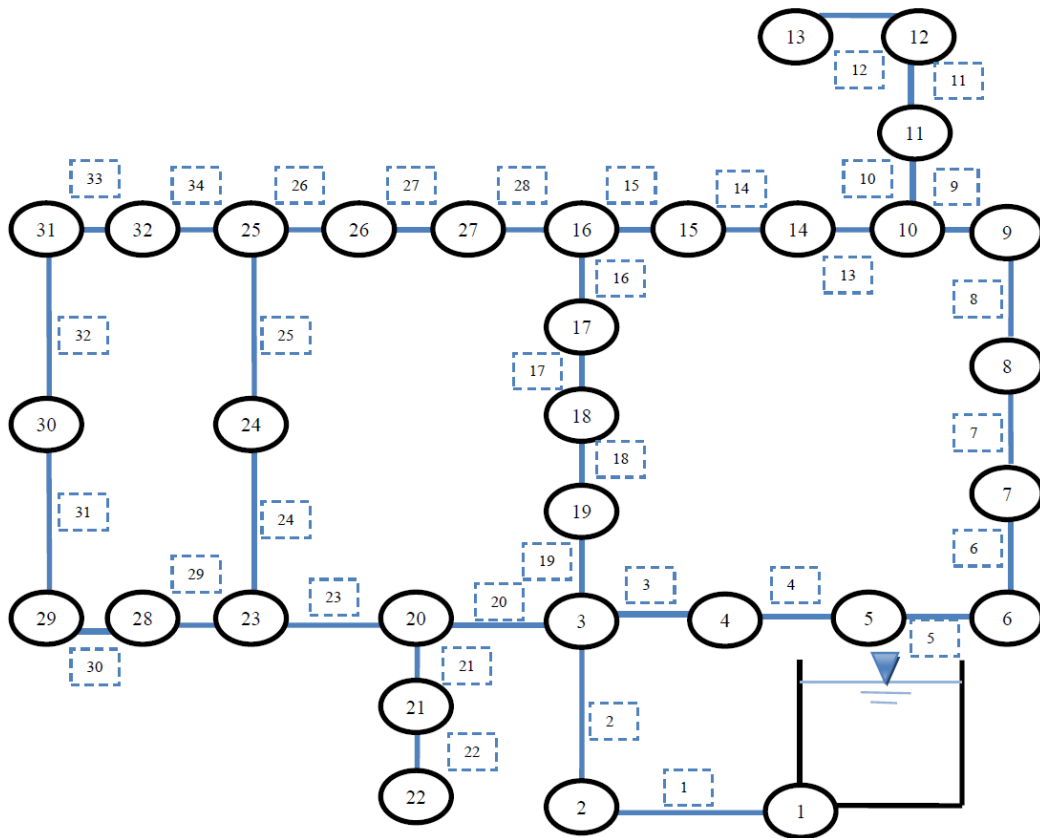
Figure 4. Hanoi water distribution network

The TWO algorithm is applied to solve this problem. Table 4 gives the results of TWO and other optimization algorithms which were previously published on this example. As shown in Table 4, the TWO algorithm has good performance than more other techniques for finding the optimal solutions. As it can be seen, the final network cost obtained by presented algorithm is equal to $5.689 million while all of the constraints are satisfied.

Table 4: Comparison of the pipe diameters and the total cost for the Hanoi network

Pipe No.	Pipe length (m)	Fujiwara	Savic and Walters	Harmony	Kaveh et al. [21]		Kaveh et al. [20]		Present algorithm (TWO)	
					Pipe length (m)	Pipe diameter (in)	Pipe length (m)	Pipe diameter (in)	Pipe length (m)	Pipe diameter (in)
1	100	40	40	40	L1=99.96 L2=0.04	D1=40 D2=30	L1=99.7 L2=0.30	D1=40 D2=30	L1=99.70 L2=0.30	D1=40 D2=30
2	1,350	40	40	40	1349.75 0.25	40 30	1347.10 2.90	40 30	1348.50 1.50	40 30
3	900	40	40	40	852.17 47.82	40 30	853 47	40 30	853 47	40 30
4	1150	40	40	40	1084.35 65.65	40 30	1084.40 65.60	40 30	1084.40 65.60	40 30
5	1450	40	40	40	1299.37 150.62	40 30	1299.70 150.30	40 30	1299.70 150.30	40 30
6	450	40	40	40	360.93	40	361.10	40	361.10	40

					89.06	30	88.90	30	88.90	30
7	850	38.16	40	40	496.46	40	496.90	40	496.90	40
					353.53	30	353.10	30	353.10	30
8	850	36.74	40	40	399.38	40	397.50	40	397.50	40
					450.61	30	452.50	30	452.50	30
9	800	35.33	40	40	224.15	40	789.40	40	714.40	40
					575.85	30	10.60	30	86.0	30
10	950	29.13	30	30	258.49	30	950	30	950	30
					691.51	24				
11	1200	26.45	24	24	1002.79	24	1200	24	1200	24
					197.2	20				
12	3500	23.25	24	24	338.32	24	1016.90	30	213.20	30
					3161.68	20	2483.10	24	3268.80	24
13	800	19.57	20	20	684.30	20	800	20	800	20
					115.70	16				
14	500	15.62	16	16	402.94	16	447.60	16	396.10	16
					97.06	12	52.40	12	103.90	12
15	550	12.00	12	12	6.99	16	550	12	550	12
					543.01	12				
16	2,730	22.50	12	12	2687.58	20	2730	16	2730	16
					42.42	16				
17	1,750	25.24	16	16	1480.29	24	1750	20	1750	20
					269.70	20				
18	800	29.01	20	20	475.23	30	800	24	800	24
					324.77	24				
19	400	29.28	20	20	246.80	30	15.30	24	188.70	24
					153.20	24	384.70	20	211.30	20
20	2,200	38.58	40	40	1573.23	40	1578.90	40	1578.90	40
					626.77	30	621.10	30	621.10	30
21	1,500	17.36	20	20	272.62	20	1500	20	1500	20
					1227.38	16				
22	500	12.65	12	12	2.82	16	500	16	500	16
					497.18	12				
23	2,650	32.59	40	40	2529.05	30	2534.90	30	2534.90	30
					120.95	24	115.10	24	115.10	24
24	1,230	22.06	30	30	1112.98	20	1111.40	20	1111.40	20
					117.02	16	118.60	16	118.60	16
25	1,300	18.34	30	30	223.13	20	222.30	20	222.30	20
					1076.87	16	1077.70	16	1077.70	16
26	850	12.00	20	20	6.01	16	850	20	850	20
					843.99	12				
27	300	22.27	12	12	299.62	20	300	12	300	12
					0.38	16				
28	750	24.57	12	12	484.67	24	704.50	16	718.50	16
					265.33	20	45.50	12	31.50	12
29	1,500	21.29	16	16	1258.09	20	1500	12	1500	12
					241.91	16				
30	2,000	19.34	16	12	848.55	20	834.80	20	834.80	20
					1151.45	16	1165.20	16	1165.20	16
31	1,600	16.52	12	12	1309.85	16	1600	16	1600	16
					290.15	12				
32	150	12.00	12	16	0.28	16	150	12	150	12
					149.72	12				
33	860	12.00	16	16	4.40	16	860	20	860	20
					855.60	12				
34	950	22.43	20	24	888.35	20	950	30	950	30
					61.65	16				
Cost ($)	-	6,320,000	6,073,000	6,056,000	5,562,343		5,741,900		5,688,700	

4.3 The Go Yang water distribution network

Kim et al. [22] originally presented the GoYang network in South Korea, as shown in Fig. 5. The system information such as elevations and water demand in each node are given in the Table 5. As the table and figure show, the system consists of 30 pipes, 22 nodes, and 9 loops, and is fed by pump (4.52 KW) from a 71m fixed head reservoir. Pipe length and their designed diameters are presented in Table 6 considering that the Hazen-Williams coefficient C is taken as 100, and 8 commercial pipe diameters that presented in Table 1 are used for this network. The minimum head limitation is assumed to be 15m above the ground level.
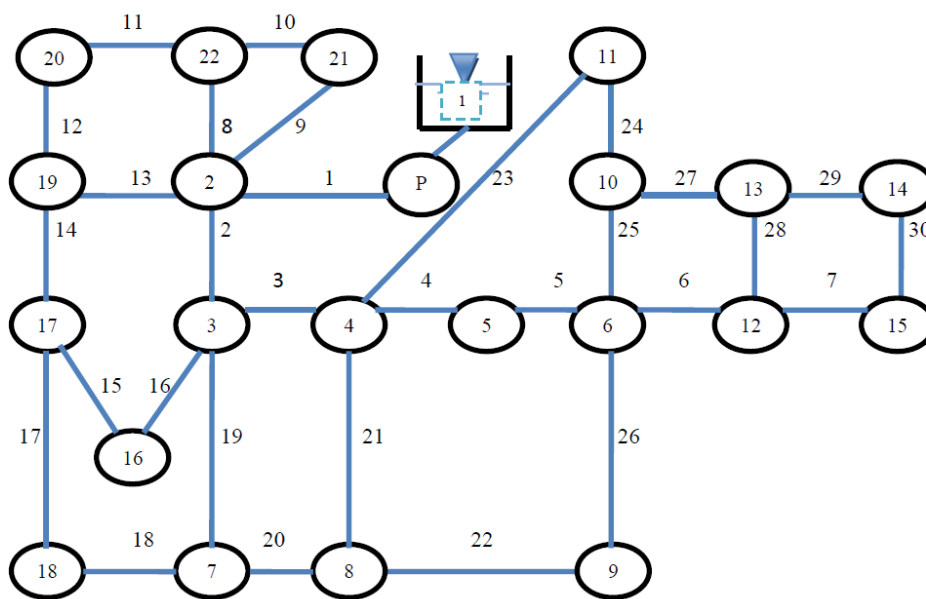

Figure 5. Go Yang water distribution network

Table 5: Nodal data and the computational results for the GoYang network

Pipe Number	Water Demand (cmd)	Ground level (m)	Pressure (original) (m)	Pressure (NLP) (m)	Pressure (HS) (m)	Pressure (CBO) (m)	Pressure (TWO) (m)
1	-2550.0	71.0	15.61	15.61	15.61	15.61	15.61
2	153.0	56.4	28.91	28.91	24.91	28.18	27.28
3	70.5	53.8	31.18	31.15	26.32	27.58	26.78
4	58.5	54.9	29.53	29.1	24.11	26.31	25.52
5	75.0	56.0	28.16	27.47	22.78	24.92	23.34
6	67.5	57.0	26.91	25.44	20.67	23.36	22.13
7	63.0	53.9	30.46	30.75	25.34	27.18	25.94
8	48.0	54.5	29.80	29.48	24.41	26.17	23.65
9	42.0	57.9	26.05	24.48	20.01	20.16	16.37
10	30.0	62.1	21.50	20.17	15.43	15.16	15.02
11	42.0	62.8	20.92	19.79	15.06	15.18	16.37
12	37.5	58.6	24.34	22.95	18.16	20.50	20.30

13	37.5	59.3	23.54	22.07	17.38	17.67	16.97
14	63.0	59.8	21.43	20.84	15.27	16.0	15.13
15	445.5	59.2	21.59	20.78	15.42	16.54	16.57
16	108.0	53.6	31.06	30.65	25.88	26.8	25.99
17	79.5	54.8	29.05	28.97	24.29	24.7	23.90
18	55.5	55.1	28.76	28.87	23.99	24.18	23.37
19	118.5	54.2	29.49	29.14	24.89	27.54	26.63
20	124.5	54.5	28.80	27.96	24.43	27.2	26.29
21	31.5	62.9	21.06	20.18	16.89	20.04	19.13
22	799.5	61.8	21.47	20.07	17.21	20.28	19.37

Table 5 shows the corresponding node pressure obtained using the TWO and other optimization methods. It can be observed that the minimum pressure limitation is satisfied in all the nodes of the network. Also, Table 6 compares the selected diameters obtained using TWO with those obtained using other methods. It is apparent that the TWO algorithm gives better results than many other methods, and the corresponding cost obtained by this algorithm is equal to 176,980,000Won (≈ $176,980), while the original cost was 179,428,600 Won (≈ $179,429).

Table 6: Comparison of the pipe diameters for the GoYang network

Pipe No.	Pipe length (m)	Diameter (original) (mm)	Diameter (NLP) (mm)	Diameter (HS) (mm)	Length (CBO) (m)	Diameter (CBO) (mm)	Length (TWO) (m)	Diameter (TWO) (mm)
1	165.0	200	200	150	L1=134.62 L2=30.38	D1=200 D2=150	L1=96.83 L2=68.17	D1=200 D2=150
2	124.0	200	200	150	108.54 15.46	125 100	111.11 12.89	125 100
3	118.0	150	125	125	0.15 117.85	125 100	35.19 82.81	125 100
4	81.0	150	125	150	15.21 65.79	100 80	73.12 7.88	100 80
5	134.0	150	100	100	120.91 13.09	100 80	90.68 43.32	100 80
6	135.0	100	100	100	113.74 21.26	100 80	128.68 6.32	100 80
7	202.0	80	80	80	202.0	80	202.0	80
8	135.0	100	80	80	135.0	80	135.0	80
9	170.0	80	80	80	170.0	80	170.0	80
10	113.0	80	80	80	113.0	80	113.0	80
11	335.0	80	80	80	335.0	80	335.0	80
12	115.0	80	80	80	115.0	80	115.0	80
13	345.0	80	80	80	345.0	80	345.0	80
14	114.0	80	80	80	114.0	80	114.0	80
15	103.0	100	80	80	103.0	80	103.0	80
16	261.0	80	80	80	261.0	80	261.0	80
17	72.0	80	80	80	72.0	80	72.0	80
18	373.0	80	100	80	373.0	80	373.0	80
19	98.0	80	125	80	98.0	80	98.0	80

20	110.0	80	80	80	110.0	80	110.0	80
21	98.0	80	80	80	98.0	80	98.0	80
22	246.0	80	80	80	10.96 235.04	100 80	97.76 148.24	100 80
23	174.0	80	80	80	174.0	80	174.0	80
24	102.0	80	80	80	55.62 46.38	100 80	50.47 51.53	100 80
25	92.0	80	80	80	40.28 51.72	100 80	19.89 72.11	100 80
26	100.0	80	80	80	100.0	80	100.0	80
27	130.0	80	80	80	130.0	80	130.0	80
28	90.0	80	80	80	18.79 71.21	100 80	42.02 47.98	100 80
29	185.0	80	100	80	185.0	80	185.0	80
30	90.0	80	80	80	90.0	80	90.0	80
Cost (Won)	-	179,428,600	179,142,700	177,135,800	176,946,211		176,980,000	

5. CONCLUSIONS

In this study, a new optimization technique, called Tug of War Optimization (TWO) algorithm, is applied to optimal cost design of water distribution networks. One of the most important features of the presented method is the simultaneous analysis, design and optimization requiring less computational time. While the analysis and optimal design of WDSs are performed in two separate phases in the existing methods (some use software such as Epanet 2, and some others employ different optimization methods). In order to show the reliability and capability in this field of engineering; three famous WDSs are considered, which have been previously studied by many other researchers. Comparisons between the performance of the TWO and those obtained by other researchers with different optimization algorithms are conducted. It is observed that optimization results obtained by the tug of war optimization method have less cost in comparison to the results obtained using other methods. Thus, in can be concluded that TWO is a suitable alternative optimizer challenging other meta-heuristic methods especially in term of optimal network design cost.

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