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Review

Surveying the Optimization Problems of Water Distribution Networks

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Abstract

This paper aims to guide understanding of the different types of optimization problems of water distribution networks by presenting a survey of mathematical models and algorithms used to solve the variants of said networks. Problems include water resource planning, water quality management, water supply networks, water distribution systems, water flow and chemical transport, and water distribution network. Optimizing resources to transport water is an issue of global interest and is important to minimize the costs of construction of pipeline to supply water, repair costs, and water transportation.

Keywords: water management, water distribution network

Introduction

Pipelines [1] transport such resources as water, petroleum products, telecommunications, chemicals, natural gas, sewage, beer, biofuels (ethanol and biobutanol), and hydrogen. The Locations are residential and commercial areas, treatment plants, processing facilities, gas stations, pumping stations, terminals, tanks, storage facilities, partial delivery stations, inlet stations, injection stations, block valve stations, regulator stations, final delivery stations, floors (levels, decks) of a building, vessels, or other structures [1]. Water transportation problems (WTP) is a generic name given to a whole class of problems in which water transportation is necessary by pipelines or other means.

Water distribution networks carry resources (drinking water, purified water, deionized water, distilled water, hard water, heavy water, soft water, tritiated water) to such locations as water storage facilities by pipelines and ducts.

A water distribution network is a hydraulic infrastructure that is part of the water supply system composed of a set of pipes, hydraulic devices (pumps, pressure-reducing valves, etc.), and reservoirs [2]. The problems that arise in the distribution of water are usually the following: distribution of cities lack adequate modernization and maintenance, which cause loss of water resources, and poor distribution of water as a consequence causes the water to run through more miles to reach its destination. Traditionally networks of water

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distribution design are done through trial and error, a process that does not have any optimization criteria, has hydraulic functionality but with rather high time, and with non-optimal results and cost. There is a need to plan networks of water distribution to be sustainable, efficient, reliable, and inexpensive. Therefore, it is necessary to determine the diameters of the pipes in the design optimally.

There are different networks of water. Among the most used in research are a New York City network composed of 20 and a tube, 20 nodes, and a circuit. The network is powered by gravity with a fixed head of 300-pound deposit. The problem that was intended to be resolved was to add new parallel to the existing pipes since the network does not meet the requirements of a load of pressure on certain nodes (for example from 16-20) [3]. The two-looped Alperovits and Shamir [4] Network consists of eight pipes, six junctions, and a reservoir. The network of water distribution of Hanoi in Vietnam [5] consists of 34 lines, three cycles, and 32 nodes, and is powered by gravity with a fixed deposit of 328 feet. The Gessler network method is based on the adaptation of the Cross method, for the flow of the network and the distribution of pressure. The method takes the heads of the nodes as main variables and sets some equations for each node with unknown headers. Every interaction relies on the heads of the front cover or it assumes initially one, and this method calculates flows and losses in the arches of the network. These flows do not satisfy the equations of continuity in the nodes, being balanced by equations of continuity [6].

Optimization Methods of Water Distribution Networks

A combinatorial optimization problem W of water distribution networks is either a minimization problem or a maximization problem and consists of the following three parts: 1) a set D_W of instances of the water distribution problem; 2) for each instance $I \in D_W$, a finite set $S_W(I)$ of candidate solutions for I; and 3) a function m_W that assigns to each instance $I \in D_W$ and each candidate solution $\sigma \in S_w(I)$ a positive rational number $m_W(I, \sigma)$ called the solution value for σ.

In the specialized literature there exist various variants of water distribution problems.

Water Quality Management

Water quality management is presented by Singh et al. [7] as a set of water sample W (N-dimensional vector), where each water sample w has a set of quality variables (1). The set of variables is presented as X, where each value of the variables is represented as x_i, where i is the specific variable (2). n is the number of variables to measure the quality of water. Equation (3) represents the I classes for each water sample w as spatial or temporal. f_c is a classification function if x_i belongs to c_j (4-5).

$$\mathrm{W} = \{\mathrm{w} | \mathrm{w} \in R^N\} \tag{1}$$

$$X = \sum_{i=1}^{n} x_i \tag{2}$$

$$C = \{c_1, c_2, \ldots, c_I\} \tag{3}$$

$$f_c\colon \mathrm{W} \rightarrow \mathrm{C} \ \exists\ x_i \in \mathrm{W} \tag{4}$$

$$f_c(x_i) = c_j \text{ if } x_i \in c_j \tag{5}$$

Water Supply Network (WSN)

WSN is an NP-hard problem [8] where we have a supply network where this problem consists of selecting the optimum design (radio or diameter) of each pipe with minimum resources cost. In this problem the author represents the demand as a graph that has a set of arcs (pipes) with different diameters. Each node needs a water supply with some flow precision represented as a nonlinear constrained optimization. WSN is represented as a water distributor P with a set of M pipes (6) and L pipe junctions (7). The initial junction is represented as Q_0, delivered with a head H_0 at the main source J_0, where the demand is presented in Equation 8. Each pipe p_i has a diameter d_i. The objective function consists of minimizing the cost C (d) from H_0 to H_j with a subset of junctions (9) subject to equations (10-11). In this case, B and K are constants based on the pipe material and cost. Equation 10 represents the commercial pipe parameters.

$$\mathrm{M} = \{\mathrm{p}_1, \mathrm{p}_2, \ldots \mathrm{p}_M\} \tag{6}$$

$$L = \{\mathrm{j}_1, \mathrm{j}_2, \ldots, \mathrm{j}_{M+1}\} \tag{7}$$

$$Q_0 = \textstyle\sum_{i=1}^{L-1} W_i \tag{8}$$

$$C(\bar{d}) = min\ C(d) = \textstyle\sum_{i=1}^{M} K_i d_i^{\beta} \tag{9}$$

$$h_j \geq H_j, \forall J_j \in F \tag{10}$$

$$d_i \in S = \{s_k\colon s_k \in R^+, k = 1,2, \ldots, N\}, i = 1,2, \ldots, M \tag{11}$$

Water Flow and Chemical Transport

Water flow and chemical transport [9] uses the formulation of Cameron and Klute and adds a first-order loss term. The equation governing the transport of a chemically reactive solute is presented in equations (12-13):

$$\frac{\partial}{\partial x_1}\left(D_{ij}\frac{\partial c}{\partial x_j}\right) - q_1\frac{\partial c}{\partial x_1} = \theta\frac{\partial c}{\partial t} + \frac{\partial(p_b c^*)}{\partial t} + f k_2 p_b c_k^* + q_5(c - c_5) \ in\ \Omega \tag{12}$$

…with

$$\frac{\partial(p_b c^*)}{\partial t} = k_1\theta c - k_2 p_b c_k^* + \frac{\partial(p_b c_e^*)}{\partial t} \quad (13)$$

…where c is the solute concentration in the liquid (in units of mass per unit volume); c^* is the adsorbed phase concentration (in units of mass of adsorbed chemical per unit mass of porous media); c_k^* is the kinetic fraction of the adsorbed chemical and $c_k^* (= k_3\theta c/p_b)$ is the equilibrium fraction; k_1 is the forward (adsorption) rate constant, k_2 is the backward (desorption) rate constant, and k_3 is the equilibrium constant; f is the loss coefficient for selective first-order removal; q_1 are the specific discharge components; p_b is bulk density; q_s is the injected/pumped fluid volume per unit aquifer volume; and C_s is the solute concentration in q_s. D_{ij} is the hydrodynamic dispersion tensor computed on the basis of and of the specific discharge and is given by equation (14):

$$D_{ij} = (\alpha_L - \alpha_T)\frac{q_i q_j}{q} + \alpha_T q \delta_{ij} \quad (14)$$

…in which α_L is longitudinal dispersivity, ar is transverse dispersivity, $q(= \sqrt{q_i q_i})$ is the magnitude of the specific discharge, and δ_{ij} is the Kronecker delta $\delta_{ij} = 1$ if $i = j$ and 0 otherwise). As mentioned earlier, this model is equivalent to the mobile-immobile physical partitioning model at equations (15-17):

$$\frac{\partial}{\partial x_1}\left(D_{ij}\frac{\partial C_m}{\partial x_j}\right) - q_1\frac{\partial C_m}{\partial x_j} = (\theta_m + p_b f * k)\frac{\partial C_m}{\partial_t} + (\theta_{im} + (1 - f^*)p_b k)\frac{\partial C_{im}}{\partial_t} \quad (15)$$

with

$$(\theta_{im} + (1 - f^*)p_b k)\frac{\partial C_{im}}{\partial_t} = \alpha_r(C_m - C_{im}) \quad (16)$$

…if we relate the parameters of these two models as:

$$C_k^* = \frac{\theta_{im}R_{im}C_{im}}{p_b};\quad k_1 = \frac{\alpha_r}{\theta_m};\quad k_2(1 - f) = \frac{\alpha_r}{R_{im}\theta_{im}};\ k_3 = \frac{f^* k p_b}{\theta_m} \quad (17)$$

…in which θ_m and θ_{im} are the mobile and immobile phases water contents; C_m and C_{im} are the mobile and immobile zone solute concentrations; f^* is the fraction of solid matrix in contact with the mobile zone; k is the distribution coefficient; α_r is first-order rate constant; and R_{im} $(= 1 + p_b(1 - f^*)k/\theta_{im})$is the retardation factor solute corresponding to the immobile zone. The similarity between the two models is made to compare the results of the numerical model with those obtained from an analytical solution of the three-dimensional mobile-immobile partitioning model.

Water Distribution Systems

The water distribution system [4] (see the mathematical model) is the dimensioning of components and the establishment of operational decisions for pumps and valves in a series of loading conditions, those that are considered "typical" or "critical." The water distribution system considers a network of water supply sources of gravity. The water distribution system (WDS) design belongs to a group of inherently intractable problems commonly referred to as NP-hard [10-11].

$$\text{Minimize}\quad C = \sum_{j=1}^{NL}\sum_{k=1}^{n(j)} C_{jk} \bullet X_{jk} \quad (18)$$

$$\sum_{k=1}^{n(j)} X_{jk} = L_j \quad \text{for all links } j \quad (19)$$

$$H_0 - \sum_{j\in p(n)}\sum_{k=1}^{n(j)} J_{jk} \bullet X_{jk} \geq H_{n\,\min} \quad \text{for all nodes } n \quad (20)$$

$$H_0 - \sum_{j\in p(n)}\sum_{k=1}^{n(j)} J_{jk} \bullet X_{jk} \leq H_{n\,\max} \quad \text{for all nodes } n \quad (21)$$

$$\sum_{j\in p'(b)}\sum_{k=1}^{n(j)} J_{jk} \bullet X_{jk} = 0 \quad (22)$$

$$X_{jk} \geq 0 \quad (23)$$

$$\sum_{k=1}^{n(j)} r_{jk} \bullet X_{jk} \leq R_j \quad (24)$$

The objective function (Equation 18) seeks to minimize the cost of the system while satisfying hydraulic criteria and reliability requirements. Optimization tries to find best diameters for network links to reach an optimum result. Within the constraints of Equation 19, length and the sum of the lengths of pipe in each link must equal the total length of the link where a link represents a pipe connecting two nodes directly. In Equations 20 and 21, head loss and the minimum and maximum permissible head at each demand point or node must be satisfied. In Equation 22, the loop, for a looped system, the total head loss around a loop must equal zero. Equation 23 represents the non-negativity. Equation 24 shows reliability and the limits of the expected (average) number of breaks in a given time period in any link. Where C_{jk} is the cost of pipe of diameter k in link j (\$/*km*), C is the total cost of the system (\$), H_{nmin} is the minimum allowable head at node n (*m*), H_{nmax} is the maximum allowable head at node n (*m*), H_o is the original head at source (*m*), J_{jk} is the hydraulic gradient for pipe diameter k in link j (*m*/*km*), L_j is the total length of link j (*km*), $n_{(j)}$ is the number of different pipe diameters in link j, NL is the total number of links within the system, $p_{(n)}$ is the links in the path from source to node n, $p_{'(b)}$ is the links in the path associated with net head loss B_p, r_{jk} is the expected number of breaks/km/year for diameter k in link j, R_j is the maximum allowable number of failures per year in link j, X_{jk} is the length of pipe of diameter k in link j (km), j is the link index, and k is the diameter type index.

Water Distribution Networks

Lansey and Mays [12] and Shamir [13] present a method to minimize the cost design of water distribution networks. Their proposal shows the uncertainties in the demands, pressure heads, and roughness coefficients. In general, reliability is defined as the probability that a system performs its mission within specified limits for a given period in a specified environment. The real issue of water distribution system reliability concerns the ability of the system to supply the demands at the nodes or demand points within the system at required minimum pressures. This methodology estimates the probability of failures base on the component failure rates. The basic optimization model for water distribution system design can be stated in general form as equations (25-29):

$$\text{Minimize} \quad \text{Cost} = \sum_{i,j\in M} f(D_{i,j}) \quad (25)$$

$$\sum_i q_{i,j} = Q_j \quad (26)$$

$$\sum_{i,j\in k} h_k = 0 \quad (27)$$

$$H_j \geq H_j \quad (28)$$

$$D_{i,j} \geq 0 \quad (29)$$

The objective function is to minimize cost as a function of diameter $D_{i,j}$ for the set of arcs, M, from nodes i to j in the network. Equation 26 shows the constraints to satisfy the demand from nodes $q_{i,j}$ (pipes flow rate from node i to node j, and Q_j represent the external demand of node j). Constraint equation 27 shows the sum of the pressure losses, h_k, around each loop $k = 1,..., K$ is equal to zero. Constraint equation 28 defines the lower bound, H_j on the pressure head, H_j, at each node. The discharge $q_{i,j}$ in each pipe connecting nodes i and j can be expressed using Hazen-Williams (Equation 29).

$$\text{Minimize} \quad \text{Cost} = \sum_{i,j\in M} \left[f(D_{i,j}) \right] \quad (30)$$

$$P\left(\sum_I K_p C_{i,j} \left[\frac{H_i - H_j}{L_{i,j}} \right]^{0.54} D_{i,j}^{2.63} \geq Q_j \right) \geq a_j \quad (31)$$

$$P\left(H_j \geq H_j\right) \geq \beta_j \quad (32)$$

$$D_{i,j} \geq 0 \quad (33)$$

The new objective function (Equation 30) is expressed regarding minimizing the costs. Constraint equation 31 is expressed as probability, P(), of satisfying demands, where K_p is a constant to account for flow units; $C_{i,j}$ is the Hazen-Williams roughness coefficient; H_i and H_j are the pressure heads at nodes i and j, respectively; $L_{i,j}$ is the length of the pipe connecting nodes i and j; and $D_{i,j}$ is the pipe diameter of the pipe connecting nodes i and j, i.e., that demand equals or exceeds probability level a_j. Similarly, constraint Equation 32 expresses the probability of the minimum pressure head being satisfied, i.e., the pressure heads equal or exceed the minimum pressure head with probability level b_j.

Di Pierro et al. [14] propose the water distribution network design for a gravitational one-loading system, as a bi-objective optimization problem with a selection of pipe sizes as the decision variables. The pipe layout and its connectivity, nodal demand, and minimum head requirements are assumed to be known. In general terms, this problem can be stated as finding the pipe sizes that minimize the total cost and head deficit while satisfying the following conditions.

- Conservation of mass: inflows and outflows must balance at each node.
- Conservation of energy: the head loss around a closed loop must equal zero.
- Head loss in each pipe is a function of the flow in the pipe, its diameter, length, and hydraulic properties (i.e., hydraulic pipe resistance).
- Pressure head and flow requirements: a minimum pressure must be provided at network locations for any given set of demands.
- Admissible pipe sizes: diameters are selected from a set of commercially available ones.

The mathematical model described by Di Pierro et al. [14] has the following formulae:

$$\begin{cases} \min\limits_{x\in\Omega} \sum_{i=1}^{D} c(x_i) \\ \min\limits_{x\in\Omega} (\max\limits_{n=1,\dots,n} \Delta P_n(x)) \end{cases} \quad (34)$$

$$\text{Subject to} \quad \sum Q_{in}^{n} - \sum Q_{out}^{n} = Q_e^n \forall n = 1,\dots,N \quad (35)$$

$$\sum_{i\in loop_j} \Delta H_i = 0 (\forall i = 1,\dots,D) \wedge (\forall j = 1,\dots,L) \quad (36)$$

$$\Delta H_i \triangleq H_i^u - H_i^d = w \frac{l_i}{CW_i^{\beta} D_i^{y}} Q_i |Q_i|^{\beta-1} \; \forall i = 1,\dots,D \quad (37)$$

$$P_n \geq P_{\min} \tag{38}$$

…where $Q \subseteq R^D$ is the decision space (that is the set of all commercially available pipe diameters); c is the cost function associated with the problem; ΔPn is the head deficit at each network node; and D, N, and L are the number of links to be designed (decision variables), the number of nodes, and the number of loops in the network, respectively. Equation (35) describes the conservation of mass while Equation (36) represents the conservation of energy, where the head loss along each pipe is given by Equation (37), with CW being the Hazen-Williams loss coefficient. Also, the pressure at each node must be equal to or greater than the minimum pressure required $P_{\min}$. From a water network design perspective, a slight pressure deficit in some non-strategic nodes (e.g., feeding non-sensitive users other than hospitals, schools, or fire-fighting hydrants) is often outweighed by the corresponding significant cost reduction. This makes a multi-objective optimization approach to water distribution design very appealing to engineers as it provides a tool to investigate interesting trade-offs.

Baños et al. [15] proposes a mathematical model of the design of water distribution networks to calculate the total cost of the network. The objective to optimize corresponding to the total cost of the network, the function is expressed as:

$$f = \sum_{i=1}^{n_d} c_i L_i + K_p \sum_{j=1}^{n_n} \max[(hr_j - h_j), 0] \tag{39}$$

…where f is the objective function to minimize, n_d is the number of existing pipe diameters, c_i is the cost per unit length of pipes from diameter i, L_i is the total length of pipelines in diameter i, K_p is the constant criminalization, n_n is the number of nodes given, hr_j is required in the node j header pressure, and h_j is the pressure of current header computed by the hydraulic simulator for node j. The value of K_p should be a very high constant (such as 100,000) to discard those solutions with pressures below the established requirements [15].

A chance-constrained optimization methodology is presented for the minimum cost design of water distribution networks. This methodology attempts to account for the uncertainties in required demands, required pressure heads, and roughness coefficients. In general, reliability is defined as the probability that a system performs its mission within specified limits for a given period in a specified environment. The real issue of water distribution system reliability concerns the ability of the system to supply the demands at the nodes or demand points within the system at required minimum pressures. This model uses component failure rates to compute component failure probabilities, which are then used to define nodal and system reliabilities. The basic optimization model for water distribution system design can be stated in general form as [16]:

$$\text{Minimize} \quad \text{Cost} = \sum_{i,j \in M} \left[f(D_{i,j}) \right] \tag{40}$$

$$\sum_i K_p C_{i,j} \left[\frac{H_i - H_j}{L_{i,j}} \right]^{0.54} D_{i,j}^{2.63} = Q_j \tag{41}$$

$$P\left(H_j \geq H_j\right) \geq \beta_j \tag{42}$$

$$D_{i,j} \geq 0 \tag{43}$$

The objective function is to minimize cost as a function of diameter $D_{i,j}$ for the set of possible links, M, connecting nodes i and j in the network. Constraint equation 41 is the continuity equation used to satisfy demand at each node in which $q_{i,j}$ is the flow rate in the pipe connecting nodes i and j, and Qj is the external demand at node j. Constraint equation 42 states that the sum of the head losses, h_k, around each loop $k = 1,..., K$ is equal to zero. Constraint equation 43 defines the lower bound, $\underline{H}_j$ on the pressure head, H_j, at each node. Discharge $q_{i,j}$ in each pipe connecting nodes i and j can be expressed using Hazen-Williams (Equation 41).

Where K_p is a constant to account for flow units; $C_{i,j}$ is the Hazen-Williams roughness coefficient; H_i and H_j are the pressure heads at nodes i and j, respectively; $L_{i,j}$ is the length of the pipe connecting nodes i and j; and $D_{i,j}$ is the pipe diameter of the pipe connecting nodes i and j. Considering the demands Q_j, the minimum pressure head requirements, H_j, and the pipe roughness coefficients, $C_{i,j}$, as random variables, the chance constrained formulation of the model. The objective function is expressed regarding minimizing the costs. Constraints are expressed as the probability, P(), of satisfying demands, i.e., that demands are equaled or exceeded with probability level a_j. They express the probability of the minimum pressure head being satisfied, i.e., the pressure heads equal or exceed the minimum pressure head with probability level b_j.

Comparison Water Distribution Network

Various algorithms to solve the variants of the water distribution networks (Table 1) may be found in the literature. We mention only some of the most popular algorithms to solve the variants of the water distribution networks.

Conclusions

Because several cities across the globe waste thousands of liters of water a day due to bad design in their water distribution networks, it is necessary to make optimal designs that minimize waste. It is important to minimize the costs of pipeline construction to supply water and

Table 1. Water distribution network comparison.

Authors	Algorithm	Problem	Contribution
Alperovits and Shamir [4]	Linear programming gradient (LPG)	Water distribution networks	The optimal design of a pipeline network that delivers known demands from sources to consumers and may contain pumps, valves, and reservoirs
Fujiwara and Khang [5]	Two-phase decomposition method	Water distribution networks	A gradient approach with the flow distribution and pumping heads as decision variables and that is an extension of the linear programming gradient method proposed by Alperovits and Shamir for the optimal design of new looped
Chu et al. [17]	Immune algorithm	WDND optimization problem	Immune algorithm and a modified immune algorithm with genetic algorithm (GA) to solve the New York City tunnel problem
Reca et al. [18]	Genetic algorithm	WDND optimization problem	The authors evaluate some meta-heuristic techniques to optimize looped water distribution systems; the genetic algorithm had the best performance for dealing with a medium-sized network
Geem [19]	Particle swarm harmony search	WDND optimization problem	A modified harmony search algorithm incorporating particle swarm concepts applied to the design of the two-loop, Hanoi, Balerma, and New York City networks
Di Pierro et al. [14]	Hybrid algorithms	Water distribution networks	Introduced ParEGO and LEMMO algorithms to solve the design problem of the network in Southern Italy and England
Baños et al. [20]	Memetic algorithm	WDND optimization problem	A computer model called MENOME with a new memetic algorithm and binary linear integer programming, a hydraulic network solver, a graphical user interface and database management module for the optimal design of looped water distribution networks
Vasan and Simonovic [21]	Differential evolution	WDND optimization problem	Application of an evolutionary optimization technique, differential evolution, a hydraulic simulation solver for optimal design of water distribution networks
Yazdani et al. [22]	Network theory approach (NTA)	Water distribution systems	Improving the resilience of water distribution systems presented as a link-node of water infrastructures and solved by network theory metrics and measurements
Chagwiza, Jones, Hove-Musekwa [23]	Max-min ant system –MMAS and a particle swarm optimization algorithm	Water distribution networks	A general mathematical programming algorithm to optimize total cost for the Bulawayo water network
Rangel et al. [24]	Artificial neural networks and genetic algorithms	Water distribution networks	Forecasting the Barcelona drinking water network in a horizon of 24 hours
Avila-Melgar, Cruz-Chávez, Martinez-Bahena [25].	Evolutionary algorithm	Water distribution networks	A methodology for using Epanet Solver with a parallel evolutionary algorithm for real life water distribution networks
Amirkhani et al. [26]	Genetic algorithms	Water distribution networks	Obtaining optimal results of the multistage operation of gated spillways for the Karkheh Reservoir in Iran
Gao et al. [27]	Genetic algorithms	Water quality	Minimizing network leakage of conventional multi-source water distributions, and applied to a real water network in Changping Town
Bozorg-Haddad et al. [28]	Honeybee mating optimization and fuzzy logic	Water distribution system	A multi-objective honeybee mating optimization and fuzzy logic to minimize the capital costs and reliability of a water distribution system
Makaremi, Haghighi, Ghafouri [29]	Genetic algorithm	Water distribution system	A non-dominated sorting genetic algorithm with the EPANET hydraulic sorting genetic algorithm to solve a real pipe network
Wang, Savić, and Kapelan [30]	Hybrid algorithm	Water distribution systems	A genetically adaptive leaping algorithm for approximation and diversity (GALAXY) to solve water distribution system
Moosavian and Lence [31]	Differential evolution (DE) algorithms	Water distribution systems	The Pareto optimal front in multiobjective problems for minimizing the cost of water distribution systems

Table 1. Continued.

Díaz, Mínguez, González [32]	Ordinary observability analysis	Water distribution networks	Computing the hydraulic state of a network from the available measurement set to compute the current status of a water distribution network
Yazdi and Moridi [33]	Soil and water assessment tool	Water quality	Models are linked to simulate the transmission and distribution of water quality variables in the Seimare watershed-reservoir system in western Iran
Pecci, Abraham, Stoianov [34]	Branch-and-bound algorithm	Water supply network	Mathematical framework for the optimal placement and operation of control valves in water distribution networks (22 nodes, 37 pipes, 3 reservoirs)
De Paola, Galdiero, Giugni [35]	Harmony search (HS)	Water distribution networks	The strategic location and setting of a prefixed number of pressure-reducing valves (PRVs)
Alexiou and Tsouros [36]	Graph theory	Water distribution networks	For determining canals to maximize the amount of water with minimum construction cost
Lence, Moosavian, Daliri [37]	Fuzzy multiobjective programming model	Water distribution systems	To identify the design pipe diameters of the water-main network of Farhadgerd, Iran
Islam, Sadiq, Rodriguez [38]	A heuristic algorithm maximum covering location problem	Water distribution networks	Maximizes the water quality index to determine cancer and noncancer risk potentials
Sebbagh et al. [39]	Genetic algorithms	Water distribution networks	The approach optimizes the parameters by means of genetic algorithms with hydraulic modeling interface, and to simulate the location of leaks in EPANET, ExpaGIS, and Optim-Detec software
Mora-Melià [40]	Different population-based algorithms	Water distribution networks	Different population-based algorithms to be applied to four well-known benchmark networks and four different algorithms
Do et al. [41]	Genetic algorithm	Water distribution systems	A genetic algorithm to calibrate the three cases of water demands for water distribution systems

repair costs, and solve water transportation problems. This survey consists of presenting the mathematical models and algorithms used to solve water distribution problems.

For future research we propose investigating mathematical models of the different kinds of flows and the creation of new algorithms that solve these optimization problems.

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