European Journal of Operational Research 228 (2013) 1–10

Contents lists available at SciVerse ScienceDirect

European Journal of Operational Research

journal homepage: www.elsevier.com/locate/ejor

Invited Review

Optimisation of gravity-fed water distribution network design: A critical review

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ARTICLE INFO

Article history:
Received 4 July 2012
Accepted 22 November 2012
Available online 3 December 2012

Keywords:
Combinatorial optimisation
Water distribution network design (WDND)
Metaheuristics

ABSTRACT

This paper surveys the literature on the optimisation of water distribution network design. The water distribution network design (WDND) optimisation problem entails finding the material and diameter of each pipe in the network so that the total cost of the network is minimised without violating any hydraulic constraints. This is a difficult combinatorial optimisation problem, in which decision variables are discrete and both cost function and constraints are non-linear. Over the past 30 years, a large number of methods, especially in the field of (meta) heuristics, have been developed to solve this problem, most of which obtain good results on the available benchmark networks. In addition to outlining the basic features of each method, a detailed computational comparison is presented. Based on this comparison, some issues with the current state of the art in this domain are discussed, and some future research directions are suggested. Additionally, the need for an adequate set of benchmark instances is motivated, and the minimal requirements for an instance set generator are discussed.

1. Introduction

A water distribution system is a network that consists of different components (pumps, reservoirs, pipes, valves, etc.) that are used to transport drinking water from one or more resource nodes to multiple demand nodes (domestic, commercial, and industrial consumers). The water must be supplied in sufficient quantities and at an adequate pressure: water that is delivered at low pressures can generally not be used, pressures that are too high can damage pipes in the network. Water distribution systems are generally owned and maintained by local governments as natural monopolies. In Flanders (Belgium), e.g., water management is organised by ten drinking water companies, that are each responsible for a certain region. International organisations such as the European Federation of National Associations of Water and Wastewater Services (Eureau) and the International Water Agency (IWA) aim to develop an international (global) drinking water policy.

Water distribution networks require decisions in four different phases with a different time horizon, as can be seen in Table 1. During the *layout* phase, the structure of the network is defined. In this phase it is determined, where the different pipes will be constructed, and where pumps, valves, water towers, reservoirs, and other components of the network should be built. In the *design* phase the type of each pipe is chosen, i.e., for each pipe in the network its optimal diameter and material are chosen. Moreover, pump types are chosen. The *programming* phase is the only phase that involves socio-economic criteria and aims to set up a priority order in which water users should be connected to the water distribution net, see Roy et al. (1992). The *planning* phase groups all decisions that are taken on a daily basis and that are concerned with the functioning of valves and operational pump levels to ensure (among many other things) that sufficient water is available in all nodes of the network, see Słowiński (1986), Burgschweiger et al. (2009) and Verleye and Aghezzaf (2011).

In this paper, a thorough review is presented of the research that has been done on the second phase, the *design* of water distribution networks. In this phase, the physical structure of the network is assumed given. With all demand nodes, resource nodes, adjacency matrix and pipe lengths assumed known, the aim of this phase is to select both the diameter and the material of each pipe in such a way that the total cost of the network is minimised and all pressure-related constraints are fulfilled. The most frequently used benchmark networks do not contain any pumps, therefore pump type optimisation is not taken into consideration in the analysed papers. This results in an optimisation problem that is both non-linear and highly constrained, and in which the decision variables are the type of each pipe (determined by diameter and material). These variables are discrete, since the type of each pipe has to be chosen from a set of commercially available types. The unit cost per meter pipe depends on the characteristics of the pipe type. As mentioned, determining pump operation levels is part of the planning phase. Therefore, these levels are considered exogenous in the design phase.

Much research has already been dedicated to the design of gravity-fed water distribution networks. To this end, a complete review of previously developed methods is provided in this paper. The remainder of this paper is organised as follows. In the next section,

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http://dx.doi.org/10.1016/j.ejor.2012.11.046

Table 1
Optimisation of water distribution networks: different phases.

Phase	Decision level	Decision variables
Layout	Strategic	System connectivity, topology, pump and valve placement
Design	Tactical	Pipe diameter, pipe roughness, pump type, etc.
Programming	Tactical/ Operational	Priority order users, etc.
Planning	Operational	Pump and valve control, etc.

the water distribution network design (WDND) optimisation problem is presented. In Section 3, an extensive overview of existing methods for the WDND optimisation problem is given. A computational comparison of these methods can be found in Section 4. Section 5 lists the main shortcomings of previous work, including the absence of an adequate set of benchmark instances. In Section 6, an overview of previous attempts to automatically generate realistic networks of arbitrary size with varying characteristics is given. Section 7 concludes and gives some pointers for future research.

2. The WDND optimisation problem

A water distribution network is usually represented as a graph in which the pipes are represented by the set of arcs P and the set of nodes N represents points of demand, points of supply (reservoirs), and junctions, i.e., points connecting two or more pipes. The objective of the WDND optimisation problem is to minimise the total cost of the network design. The cost of an individual pipe depends on the *type* t that is chosen for this pipe from a list of commercially available types T. The type of a pipe determines both its diameter and the material of which it is made, which in term determine its hydraulic properties. If the cost per meter of a pipe of type t is represented by C_t and the length of pipe p is represented as L_p, the objective function of the WDND optimisation problem can be written as:

$$\text{Minimise total cost} = \sum_{p \in P} \sum_{t \in T} x_{tp}.C_t.L_p \tag{1}$$

where x_{tp} is a binary decision variable that determines whether pipe p is chosen from type t ($x_{tp} = 1$) or not ($x_{tp} = 0$).

The objective function is conditioned by physical mass and energy conservation laws, and by minimum head requirements in the demand nodes.

The *mass conservation law* must be satisfied for each node $n \in N$. This law states that the volume of water flowing into a node in the network per unit of time must be equal to the volume of water flowing out of this node. Let Q_{ij} represent the water flowing from node i to node j, and let S_n be the supply and D_n the demand of node n (all expressed in cubic meter per second) then the following should hold:

$$\sum_{i \in N \setminus n} Q_{in} - \sum_{j \in N \setminus n} Q_{nj} = D_n - S_n \quad \forall n \in N \tag{2}$$

Furthermore, for each closed loop $l \in O$, the *energy conservation law* must be satisfied. This law states that the sum of pressure drops in a closed loop is zero. Pressure drops (also called head losses) in piping systems are caused by wall shear in pipes and friction caused by piping components such as junctions, valves, and bends. In past research, only the first type of friction losses (in the pipes) are taken into account. Let ΔH_p represent the head loss in a pipe p (in m) that connects nodes n_1 and n_2, then:

$$\Delta H_p = H_{n_1} - H_{n_2}$$

For the closed loop l, the energy conservation law can therefore be stated as:

$$\sum_{p \in \text{loop } l} \Delta H_p = 0 \quad \forall l \in O \tag{3}$$

Approximating the head losses in the pipes of the network is usually done using the Hazen–Williams approximation:

$$\Delta H_p = w \frac{Q^a L_p}{C^a D^b} \tag{4}$$

In this equation, Q is the water flow rate in (cubic meter per second), L_p is the pipe length (in meter), C is the Hazen–Williams roughness coefficient (unitless), D is the pipe diameter (in meter), and a, b, and w are unitless parameters. Parameters D and C are determined by the type of a pipe and are assumed given for each available type. An example of both the mass and the energy conservation law can be seen in Fig. 1.

Finally, *minimum pressure head requirements* exist for every (demand) node $n \in N$. Let H_n be the pressure head in node n (in meter) and $H_n^{\min}$ the minimum pressure head in node n (in meter). This constraint therefore can be represented as:

$$H_n \geqslant H_n^{\min} \quad \forall n \in N \tag{5}$$

In recent years, some authors have started to include additional requirements in the optimisation problem. Geem et al. (2011), e.g., add minimum and maximum water velocity constraints to increase the reliability of the network.

The WDND optimisation problem assumes that pipe layout, connectivity, nodal demands, and minimum head requirements in the nodes are known and constant. Although water distribution networks are dynamic systems, in which nodal demands and head requirements evolve over time, most research has focused on this static problem. Other components frequently found in real-life water distribution networks, such as pumps, valves, and water towers, are also disregarded in research on the WDND optimisation problem.

Most papers use the hydraulic solver EPANET2.0 to check hydraulic feasibility of the solutions they generate, although some papers use the previous version (EPANET1.0). EPANET software employs the gradient method proposed by Todini and Pilati (1987) to solve the mass and energy conservation laws. The head loss equation parameters in version 2.0 of EPANET are $a = 1.852$; $w = 10.6668$ (for SI units) and $b = 4.871$. The Hazen–Williams expression then reduces to:

$$\Delta H_p = 10.67 \frac{Q^{1.852} L_p}{C^{1.852} D^{4.871}} \tag{6}$$

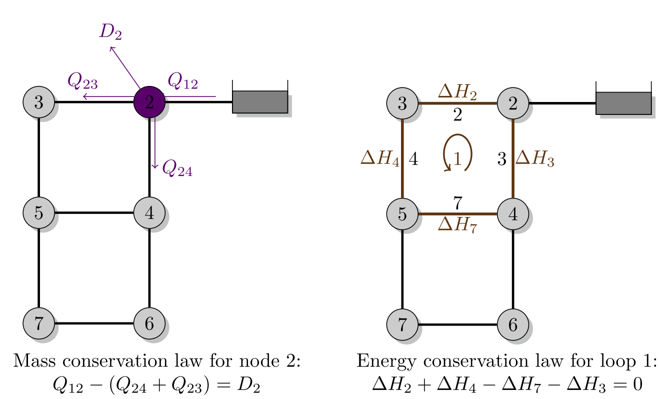

Fig. 1. Example of the mass (left) and the energy conservation law (right) applied to the two loop network of Alperovits and Shamir (1997).