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The influence of the existing network layout on water distribution system redesign analysis

C. Tricarico, M. S. Morley, R. Gargano, Z. Kapelan, G. de Marinis and D. A. Savić

ABSTRACT

The methodologies usually adopted in water distribution system redesign problems consider the topology of the network as an input fixed datum; optimisation solely allowing for the duplication/substitution of existing components. In order to contribute to the identification of optimal solutions that may lead to a lower risk of failure to supply the required water, together with a lower redesign cost, this paper proposes a novel methodology which reports the influence of the existing network configuration and its performance. In particular, the redundancy of loops and the robustness of the network topology are investigated by applying an optimisation technique based on a genetic algorithm and by taking into account the random water demand at each node. The methodology presented has been applied to two case studies, in which it considers the influence of the topology on the overall system reliability/risk. The results demonstrate that it is possible to obtain further configurations that are more reliable for a lower redesign cost. The analysis performed highlights the impact of the topology on the search for an optimal solution, which, as a principal conclusion of the work, should be considered among the decision variables taken into account by the optimisation in a redesign problem.

Key words | genetic algorithm, multiobjective optimisation, network topology, redesign, risk-cost, water distribution systems

C. Tricarico (corresponding author)
R. Gargano
G. de Marinis
Dipartimento di Ingegneria Civile e Meccanica, Università degli Studi di Cassino e del Lazio Meridionale,
via Di Biasio, 43,
Cassino,
Italy
E-mail: *c.tricarico@unicas.it*

M. S. Morley
Z. Kapelan
D. A. Savić
Centre for Water Systems,
University of Exeter,
Harrison Building,
North Park Road,
Exeter,
Devon,
UK

INTRODUCTION

Nowadays, much focus of urban hydraulic infrastructure design, in developed countries, relates to the redesign of existing systems. Often the urban water distribution networks are unable to satisfy water requirements, themselves increasing, while the performance of the system itself deteriorates, owing to its ageing infrastructure. However, the funding available for undertaking the necessary improvements is usually insufficient to accommodate all of the required works to fully rehabilitate the system. On the basis of this consideration, a trade-off exists between the need to improve the performance of a system and to limit the redesign costs to the available funds. For this purpose, it is necessary to determine the optimal improvements that can be made to the network infrastructure which relate to the minimum possible cost while simultaneously demonstrating the maximum reliability.

Accordingly, the application of the redesign optimisation problem to an existing network is of noteworthy interest, as demonstrated by the numerous contributions on the topic in the technical literature (e.g., Kapelan *et al.* 2006; Giustolisi *et al.* 2007; Arena *et al.* 2010; Yoo *et al.* 2010; Marchi *et al.* 2014).

However, redesign/rehabilitation techniques have usually considered the existing topological scheme as an input datum and constrained the problem of searching for the optimal solutions as being among those which allowed the duplication/substitution of the existing components: pipe diameters, electromechanical components, modified

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pump operation, tank levels, etc. (e.g., Tricarico *et al.* 2006; Berardi *et al.* 2008; Alvisi & Franchini 2009; Morley *et al.* 2012). In particular, in reference to the topology of the system, rehabilitation strategies are not considering the possibility of modifying the existing layout, leaving it at most for the design of new expansion zones (e.g., Halhal *et al.* 1997).

Thus, although there are several algorithms which take into account the variability of the topological scheme as part of the optimisation process (e.g., Rowell & Barnes 1982; Goulter & Morgan 1985; Goulter 1987; Jacobs & Goulter 1989; Tolson *et al.* 2004; Ostfeld 2005; Afshar & Jabbari 2008) these do not take into consideration the variation of the network layout for the purposes of rehabilitation/ redesign.

Nevertheless, the initial water distribution system (WDS) topology, if treated as an input datum and not as a decision variable in itself, could influence the search, leading the algorithm to identify solutions which, even if part of a Pareto front, do not represent the optimal configurations in terms of the trade-off between cost and reliability (Tricarico *et al.* 2012). Furthermore, they may also be shown to be less than sensible solutions from an engineering point of view.

This paper presents a methodology, novel compared to the classical analysis that is usually undertaken in the literature for a redesign problem, in which the existing network topology is considered as a decision variable and can be changed by considering the possibility of removing, if necessary, existing components or through adding new ones to the initial configuration. This approach, coupled with the optimisation process of duplicating or substituting the original components, can lead to a revised redesign problem that tends towards solutions that may prove more reliable and, at the same time, less costly.

This approach has been applied using a multiobjective optimisation model, employing an extension of the economic level of reliability (Tricarico *et al.* 2006): a cost–risk analysis (de Marinis *et al.* 2008) in which the objectives considered are the minimisation of the risk associated with water demand satisfaction and the minimisation of redesign cost (economic level of risk (ELK)). The optimisation is undertaken through the application of a modified version of omni-optimiser (Deb & Tiwari 2008) with revised constraint handling and additional local search operators and a pressure-driven hydraulic solver (Morley & Tricarico 2008). Demand at network nodes has been considered as an uncertain variable and has been modelled by means of a probabilistic approach. In this fashion, the search of the optimal configurations has been undertaken by taking into consideration solely the hydraulic reliability of the systems: considerations of mechanical reliability or leakage analysis have not been addressed.

The study of the influence of the network layout on optimal redesign configurations has been undertaken by comparing the application of a redesign algorithm in two different approaches. First, the algorithm has been applied to the network by considering the ‘classical’ redesign approach in which the network layout is predetermined, allowing solely the possibility of duplicating or substituting the existing pipes *in situ*. Subsequently, the algorithm has been re-run, instead permitting the freedom in deciding the network topology by modifying the layout through the closure of existing pipes and adding new links between nodes, in addition to the possibility of duplicating or substituting the existing pipes. The candidate links to be added between nodes are all the possible physical connections which can be sensibly realised from an engineering point of view.

The Pareto fronts obtained from the aforementioned optimisation approaches have been compared and the results demonstrate the importance of considering the layout of the network as a decision variable in a redesign problem in which the optimal solution may be characterised by a topological scheme that differs from the existing system. Furthermore, as a function of the network analysed, it is possible also to detect a specific *set* of loops which can characterise the optimal solution. Layouts with further loops with respect to the one detected from the *set* are representative of solutions dominated by the Pareto front obtained. Of course ‘where’ such a set of loops is formed strictly depends on the system analysed and the water demand at network nodes.

The methodology adopted for the case study presented in this work considers solely the pipes as decision variables. However, the other potential components of the network, such as tanks, pumps, valves, etc. could be added as options without invalidating the proposed approach.

OPTIMISATION PROBLEM

The WDS redesign problem is formulated here as a multi-objective optimisation method under uncertain demands. The characterisation and quantification of uncertainty in water demands is achieved by means of a probabilistic approach: the water consumption required at each network node is modelled as an independent random variable distributed with a predefined probability density function. The parameters and the type of probability distribution adopted are derived through experimental analysis performed on real-life distribution networks. In particular, the peak water demand being the condition of principal interest in a redesign of a hydraulic system, the log-normal distribution has been shown to be effective in modelling the nodal demand variation (e.g., Tricarico *et al.* 2007; Gargano *et al.* 2010). The mean has been considered equal to the base demand at each network node and the CV value has been assumed equal to 0.1, considering a number of users greater than 1,000 (Gato-Trinidad & Gan 2012).

The objectives considered are: (1) minimisation of the total redesign cost C_{Tot} (sum of the structural costs C_{ST} and the lost revenue costs C_{LR}); and (2) the minimisation of the 'network risk' K_{Net} – estimated herein as the maximum of the risks evaluated at each node of the network, K_i. In this fashion the model searches the optimal solutions among the configurations which present low risk in each node. However, the 'network risk' could be also estimated by considering the sum or the average of the nodal risk without this affecting the methodology suggested. The K_i has been herein considered as the product of the probability of failure in satisfying the water demand and the consequence of this failure. The damage caused to the customers by failing to satisfy the full demand has thus been estimated by considering solely the water volume shortfalls estimated by the hydraulic solver. Socioeconomic considerations related to it, such as the user types (e.g., residential users, hospitals, public offices, shopping centres) and an estimation of the consequent economic damage in failing to supply the water requested, have not been addressed in this work along with a frequency analysis of the shortfalls which would require an extended period simulation approach to be adopted. Therefore, the risk for each node i of the system (K_i) is estimated by the following expression:

$$K_i = (1 - R_i)\left[\frac{\sum_{s=1}^{N_{\mathrm{S}}} Q_{\mathrm{REQ}_i s} - \sum_{s=1}^{N_{\mathrm{S}}} Q_{\mathrm{DEL}_i s}}{\sum_{s=1}^{N_{\mathrm{S}}} Q_{\mathrm{REQ}_i s}}\right] \quad \forall i \in \{1, \ldots, N_{\mathrm{n}}\}, \tag{1}$$

where $Q_{\mathrm{REQ}i}$ and $Q_{\mathrm{DEL}i}$ are, respectively, the water required and the water delivered at the ith node, N_{n} is the total number of nodes in the WDS and N_{S} represents the number of samples generated by means of the probabilistic approach.

The first factor of Equation (1) (1–R_i) represents the failure probability; R_i is the hydraulic reliability of the system node ith, defined as the probability of meeting the water requirements for that network node and it is evaluated considering the water effectively supplied to the customers as a function of the available nodal heads:

$$R_i = \frac{\sum_{s=1}^{N_{\mathrm{S}}}\left(Q_{\mathrm{DEL}_{i,s}}/Q_{\mathrm{REQ}_{i,s}}\right)}{N_{\mathrm{S}}} \quad \forall i \in \{1, \ldots, N_{\mathrm{n}}\} \tag{2}$$

while the consequence of this failure, i.e., the damage index, is addressed in the second factor of Equation (1).

With these considerations, the stochastic multiple objective optimisation problem formulation is as follows:

$$\text{Minimise } C_{\mathrm{Tot}} = C_{\mathrm{ST}} + C'_{\mathrm{LR}}, \tag{3}$$

$$\text{Minimise } K_{\mathrm{Net}} = \max\{K_i\}. \tag{4}$$

Subject to:

- Hydraulic equation constraints:

$$\sum_{j=1}^{Nj,i} q_j - Q_{\mathrm{DEL}i} = 0 \quad (i = 1, \ldots, N_{\mathrm{n}}), \tag{5}$$

$$H_{j,\mathrm{u}} - H_{j,\mathrm{d}} = r_j \cdot q_j^{\varepsilon} \quad (j = 1, \ldots, N_{\mathrm{l}}), \tag{6}$$

where q_j is the flow in the jth pipe; $H_{j,\mathrm{u}}$, head at upstream node of the jth pipe; $H_{j,\mathrm{d}}$, head at downstream node of the

jth pipe; r_j, coefficient of the jth pipe (headloss formula, function of pipe length, diameter and roughness coefficient); ε is the flow exponent function of the headloss formula used; $N_{j,i}$ is the number of pipes connected to the ith network node; N_l is the number of network links; and N_n the number of network nodes.

- Decision variable constraint:

$$D_k \in D \ (k = 1, \ \ldots, N_d), \tag{7}$$

where D_k is the value of the kth discrete decision variable (available diameters); D is the discrete set of available redesign options; and N_d is the number of decision variables.

- The first objective considered, total redesign cost C_{Tot}, is given by the summation of the capital intervention (i.e., structural) cost C_{ST} – a function of the redesign works – and the lost revenue costs C_{LR} – a function of the network water volume not delivered (Tricarico *et al.* 2006).

In order to combine the *operational* lost revenue costs with the *capital* structural redesign costs, a conversion is required. The annual value of the lost revenue C_{LR} is thus converted into its net present value as follows:

$$C'_{LR} = C_{LR} \cdot [1 - (1 + p)^{-T}]/p, \tag{8}$$

where p is the discount rate and T is the assumed system lifespan in years.

The optimisation employs the multiobjective omni-optimiser algorithm (Deb & Tiwari 2008). This algorithm belongs to the genetic algorithm (GA) family of population-based evolutionary algorithms and has been successfully employed in a number of WDS optimisation applications (e.g., Vamvakeridou-Lyroudia *et al.* 2009; Morley *et al.* 2012) and is well suited to applications involving both discrete integer and real decision variables.

This algorithm has been augmented with the robust optimisation technique introduced by Kapelan *et al.* (2006). This approach employs a concept of nested, in-process sampling that allows greater confidence in the range of objective values when considering uncertain parameters – in this case, system demand. The related, robust non-repeating GA (Morley 2008), which has been employed successfully on similar problems hitherto, was not applied in this instance as the population diversity strategy employed by omni-optimiser militates against the effective use of such an archiving technique.

In order to estimate the objectives of the optimisation problem and thus the potential failure of the network in supplying the water required by users, a pressure-driven hydraulic solver, in which the outflow from the network is assumed to be a function of the heads available at each of the nodes, has been adopted in order to model the system behaviour (e.g., Todini 2003; Giustolisi *et al.* 2008).

Specifically, an extension of the core EPANET library, EPANETpdd (Morley & Tricarico 2008) is employed, derived from two existing modifications: an extension to EPANET presented by Cheung *et al.* (2005) which adds pressure-driven demand to the OOTEN (object oriented toolkit for EPANET) toolkit of van Zyl *et al.* (2003) provided by the University of Johannesburg and a revised pressure-driven version of EPANET (Rossman 2007) obtained from its author. EPANETpdd combines the robust hydraulic simulator facilitated by the latter and the constrained emitter behaviour of the former. Significant modifications in the implementation have also been added. First, EPANETpdd allows the specification of variable critical pressure ($H_{critical}$) values at which 100% of the required demand can be considered to be delivered for each node of the network. In addition, the exponent α of the orifice control formulation can be specified on a per-node basis in order to give a greater control of a parameter that is difficult to estimate (Ackley *et al.* 2001) and thus permitting a greater flexibility in the description of a hydraulic network problem. In addition, the resulting pressure-driven hydraulic solver does not require the specification of the outflow in terms of the emitter coefficient c which is, instead, dynamically calculated according to the demand specified in the input file – simplifying the conversion of a demand-driven model to a pressure-driven one while also facilitating the use of extended period simulation (EPS) under these conditions.

CASE STUDIES

In order to highlight the effect of the topological scheme on WDS performance, the redesign methodology presented

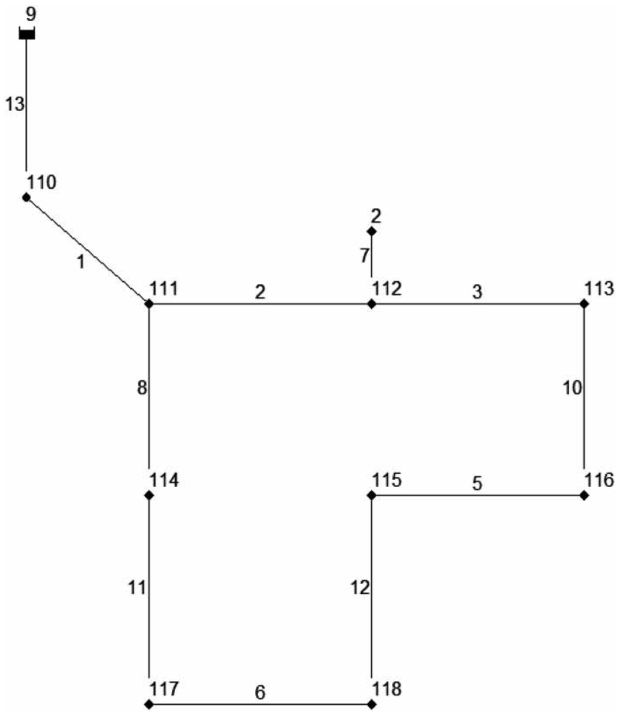

Figure 1 | Network A topology.

above has been applied to two synthetic case studies. The optimisation methodology has been applied by considering the *classical redesign approach* in which the existing pipes' layout is considered as an input datum and the results have been compared with the *novel methodology* in which the topology of the network is considered variable.

The input data considered in the optimisation model and applied to the case study considered are herein

Table 1 | Node characteristics – Network A

Node ID	Elevation [m]	*Q* [l/s]
110	205	–
111	205	35.0
112	202	35.0
113	200	22.5
114	202	33.75
115	200	45
116	199	33.75
117	202	22.5
118	202	22.5
2	215	35.0

Table 2 | Pipes characteristics of Network A (a) and possible new connections A-AC (b); for any pipe a roughness value of 100 evaluated with the Hazen–William (H–W) relationship is considered

(a)

Pipe	*L* [m]	*D* [mm]
1	1,609	457
2	1,609	356
3	1,609	254
5	1,609	305
6	1,609	152
7	1,609	457
8	1,609	254
10	1,609	203
11	1,609	203
12	1,609	152
13	1,609	152

(b)

Pipe	*L* [m]
4	1,609
9	1,609
9_111	1,300
9_112	2,900
9_2	2,900
9_113	4,400
9_116	3,900
9_114	2,900
9_117	3,500
9_118	3,900
2_110	2,800
2_111	1,610
2_113	1,610
110_112	2,800
110_113	3,400
110_116	3,800
110_117	3,400
110_118	3,800
110_114	2,800
113_118	1,600
116_118	1,300

reported. Multiple GA runs were performed using several different initial populations (i.e., many different random seeds), employing 20 Latin hypercube samples and a minimum chromosome age of 20 generations. In all runs, a population size of 250 was used and run for 1,500 generations. Following the optimisation runs, all of the solutions obtained were re-evaluated using 100,000 Monte Carlo samples, in order to take into account the uncertainty in the water demands. In order to apply the pressure-driven methodology, a minimum pressure requirement of 20 m has been adopted for all demand nodes in all cases. The lost revenue estimation has been undertaken using, as an example, a unit cost of water of 1 €/m^3 and, in order to compare these costs with those of the infrastructure, a system lifespan of 50 years and an interest rate of 3.2% (an average value at the European level) have been used in the relevant calculations.

The first synthetic case study examined, called Network A, is characterised by 1 loop, 11 pipes, 10 nodes and a single reservoir at node 9 as shown in Figure 1. The head of the reservoir at node 9 has been assumed fixed at 280 m. The input characteristics considered in the optimisation model are reported in Tables 1 and 2(a).

The simplicity of case study Network A allows an effective understanding of the importance of the topological scheme as decision variable in an optimisation to rehabilitate an existing network.

The possible redesign solutions for each of the potential 11 new pipes in the network have been considered, first, in a classical redesign approach in which the optimisation may do nothing or elect to install a new parallel pipe with one of the 18 new pipe diameters available (Table 3). The total number of possible solutions, i.e., network configurations, is thus equal to 19^{11} (i.e., 1.16×10^{14}).

The redesign problem has been then solved for the same network by considering as decision variables not only the possibilities for the existing pipes, but also the possibility of closing existing pipes or adding extra links (i.e., new pipes) between nodes with a choice of possible diameters as reported in Table 3. All of the possible extra links have been assigned as decision variables prior to the optimisation process (Figure 2 – Network A-AC and Table 2(b)). Any connection that was not previously existent in the original layout between nodes is considered possible. Of course, from a practical engineering standpoint, the modeller should identify the appropriate, additional connections among nodes to avoid superfluous links being considered. In this case, thus, the total number of possible solutions,

Table 3 | Available diameters for duplication and relative costs (diameter data reported refer to PEAD PN10 material; relative costs are average values taken from an Italian price list)

DN	75	90	110	125	140	160	180	200	250	280	315	355	400	450	500	630	710	800
$D_{internal}$ [mm]	66	79.2	96.8	110.2	123.4	141	158.6	176.2	220.4	246.8	277.6	312.8	352.6	396.6	440.6	555.2	625.8	705.2
C [€/m]	100	105	118	123	133	141	150	163	194	212	241	269	303	345	407	562	707	852

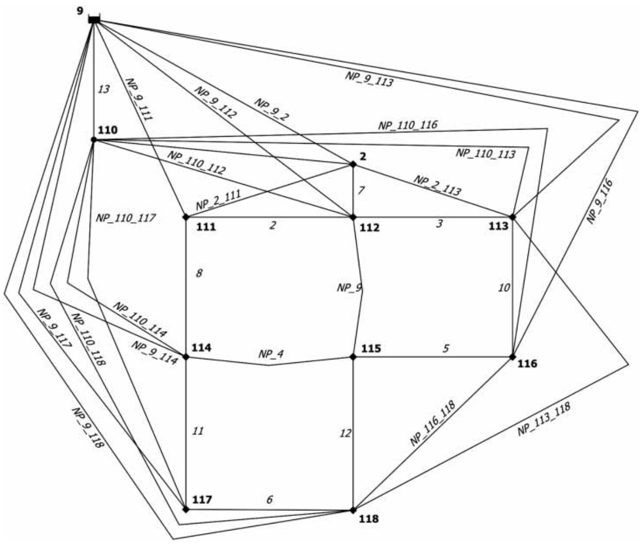

Figure 2 | Network A-AC topology.

i.e., network configurations, is equal to $19^{11} \times 19^{21} = 19^{33}$ (i.e., 1.58×10^{42}). Furthermore, by considering the possibility of closing existing connections as well, the total number of possible configurations in this novel methodology becomes $19^{33} \times 2^{11}$ (i.e., 3.24×10^{45}).

The analysis, applied to the Network A problem, may allow a rapid and immediate analysis of the influence of the network topology on the cost–performance redesign solutions. The resulting Pareto fronts are reported in Figure 3.

As one might expect, as the total costs of redesign increase, the potential risk of failing to supply the water required to the customers reduces. By means of the extension to the risk of the cost–reliability optimisation (Tricarico *et al.* 2006) it is possible to note that the optimal configuration which has been circled in Figure 3, is related to that particular solution of the Pareto front which corresponds to the minimum total redesign cost, i.e., the minimum structural cost and minimum lost revenue (through failure to supply). This solution, the ELK, according to social and technical considerations, can be seen as the optimal configuration to be adopted in a redesign program or at least as a threshold solution which helps the decision-maker in selecting which of the redesign configurations to adopt among those solutions having lower risk values than previously identified. Through the comparison of the two Pareto optimal fronts determined with the two methodologies (Figure 3), it is possible to note how allowing the variation of the system layout leads to solutions which dominate the classical approach in terms of redesign costs and risk. However, for this case study, a significant difference between the two Pareto fronts is evident for solutions with lower values of risk (i.e., $K_{\mathrm{Net}} \leq 0.04$).

The Net A-AC Pareto front is characterised by solutions with different looped configurations for various risk values. However, even if, as expected, increasing the

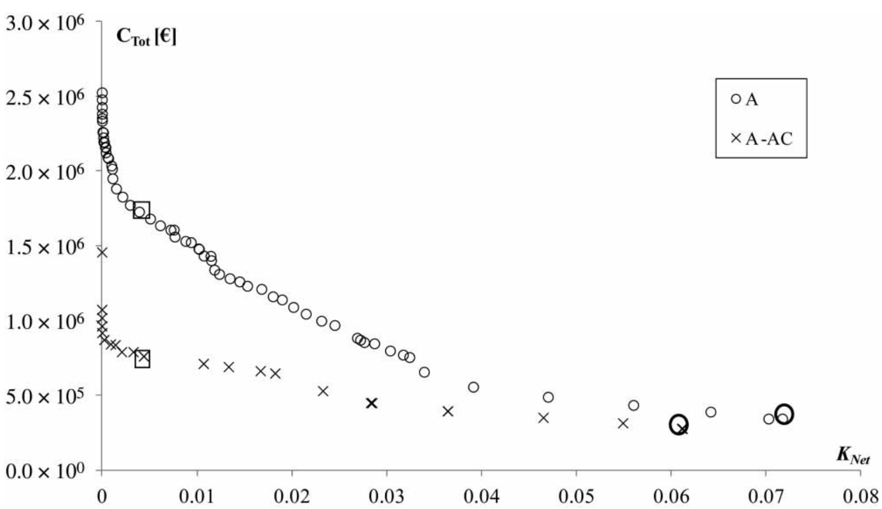

Figure 3 | Pareto fronts in comparison between classical and novel methodologies for Network A.

number of loops in the network reduces the risk of not supplying the required water, the Net A-AC solutions obtained do not reveal configurations with more than three loops. This result highlights that for the network under consideration, a set of three loops can be considered the optimal configurations and a further increase in loops does not result in a corresponding improvement in system performance or cost. This result suggests the presence of a threshold in redundancy, i.e., in the number of loops, for such systems in order to get a reliable network with lower redesign costs. On one hand, the looped network scheme is important because, ordinarily, the more loops present, the more reliable the network. On the other hand, the result obtained for the case study highlights that it is economically convenient to have a 'threshold' on the redundancy of the system in order to identify an optimal configuration.

In particular, a cost analysis of the ELK Pareto front solutions obtained by means of the two methodologies is reported in Table 4. It is shown that the ELK solution for Net A-AC has a lower risk value with furthermore a lower redesign cost. This suggests that by applying the novel methodology, the resulting configuration is more reliable even with a lower redesign cost. Initial network configurations with a single loop have to be strengthened further, at a commensurately higher total redesign cost, to reach the same level of risk. However, this logical result posits the importance of modifying the existing layout in order to obtain more reliable and less costly configurations.

A total redesign cost comparison is reported in Table 5, for solutions with the same level of risk ($K_{\mathrm{Net}} = 0.004$). These solutions are those highlighted by squares in Figure 3. As can be seen, the classical redesign method needs to duplicate more pipes in order to reach an equivalent level of risk with respect to the solutions obtained with the novel methodology in which the addition of merely

Table 4 | ELK comparison between Network A and Network A-AC

	K_{Net}	C_{Tot} [€]
A (1 loop), Classical methodology	0.07	343,017
A-AC (1 loop), Novel methodology	0.06	277,273

Table 5 | Cost analysis of the Pareto front solutions with the same network performance (a) and relative solutions for Network A and Network A-AC (b)

	K_{Net}	C_{Tot} [€]
(a)		
A (1 loop), Classical methodology	0.004	1,720,000
A -AC (1 loop), Novel methodology	0.004	760,767

(b)												
A (1 loop), Classical method	Pipe ID	1	2	3	5	6	7	8	10	11	12	13
	DN	–	–	315	–	–	–	–	355	–	–	630
A-AC (2 loops), Novel method	Pipe ID	1	2	3	5	6	7	8	10	11	12	13
	DN	–	–	–	closed	–	–	–	–	–	–	–
	Pipe ID	NP4	NP9	NP9-111	NP9-112	NP9-2	NP9-113	NP9-114	NP9-116	NP9-117	NP9-118	NP2-110
	DN	–	250	450	–	–	–	–	–	–	–	–
	Pipe ID	NP2-111	NP2-113	NP110-112	NP110-113	NP110-114	NP110-116	NP110-117	NP110-118	NP113-118	NP116-118	
	DN	–	–	–	–	–	–	–	–	–	–	

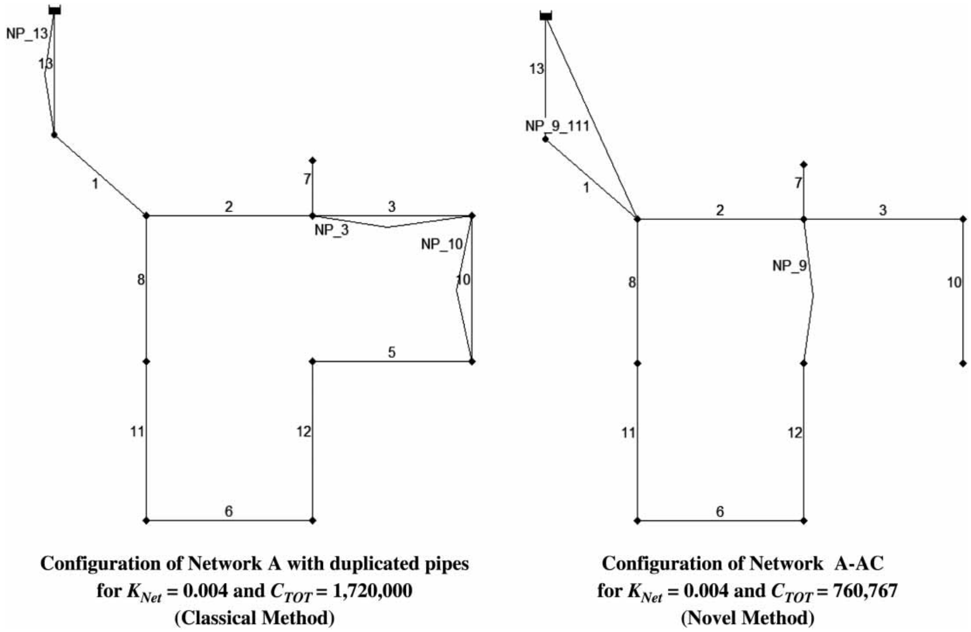

Figure 4 | Network A topologies for configurations presented in Table 5.

one or two pipes results in a high robustness with a considerable reduction in the necessary capital costs. The network configurations corresponding to those of this comparison are illustrated in Figure 4.

Analogous results have been further obtained by applying the same methodology on a more complex case study: Network AT (Figure 5). This is composed of 19 pipes (three loops), 16 nodes and one reservoir, for

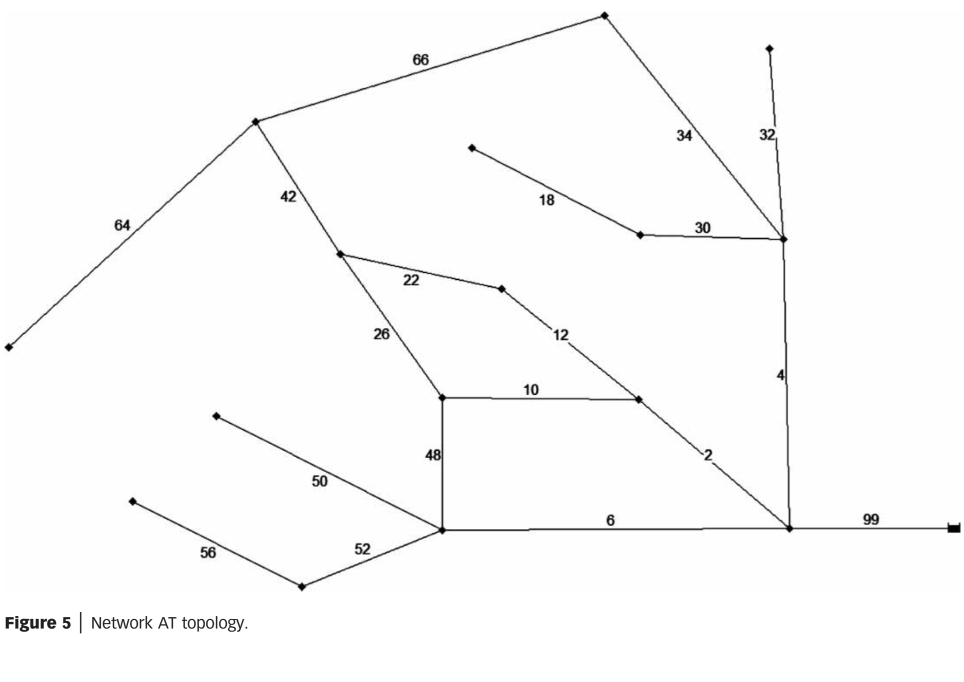

Figure 5 | Network AT topology.

Table 6 | Node characteristics – Network AT

Node ID	Elevation [m]	Q [l/s]
20	6.096	15.68
30	15.24	12.27
40	15.24	12.27
50	15.24	16.81
60	15.24	15.68
70	15.24	15.68
80	15.24	15.68
90	15.24	11.36
100	15.24	15.68
110	15.24	15.68
120	36.58	14.54
130	36.58	14.54
140	36.58	14.54
150	36.58	14.54
160	36.58	11.36
170	36.58	14.54

which the head has been fixed at 58 m. The input characteristics of the AT network are reported in Tables 6 and 7 (a). The number of duplication pipe diameters available is shown in Table 3 (18 new pipe diameters available), thus the total number of possible solutions, i.e., network configurations, in the classical redesign approach, in which the optimisation may do nothing or elect to install a new parallel pipe with one of the 18 new pipe diameters available, is therefore equal to $19^{19} \approx 1.98 \times 10^{24}$. In order to determine the potential influence of the layout scheme on the cost–risk analysis, several possible new connections have been considered, as reported in Figure 6: Network AT-AC. The input characteristics of Network AT-AC are reported in Tables 6 and 7. The possible pipe diameter options remain those shown in Table 3, thus with the novel methodology, the total number of possible network configurations – being possible to duplicate the 19 existing pipes, adding 22 new connections and closing the 19 existing connections – are equal to $19^{19} \times 19^{22} \times 2^{19} \approx 1.41 \times 10^{58}$.

The Pareto fronts obtained in comparison between the two Networks AT and AT-AC (Figure 7) again show that a reduction in risk corresponds, as might be

Table 7 | Pipes characteristics of Network AT (a) and possible new connections A-AC (b)

(a) Pipe	L [m]	D [mm]	Rough [H–W]
2	3,658	406.4	70
6	3,658	304.8	70
10	1,829	304.8	70
12	1,829	254.0	70
18	1,829	304.8	70
22	1,829	254.0	70
26	1,829	304.8	70
48	1,829	203.2	70
4	3,658	304.8	120
30	1,829	254.0	120
32	1,829	254.0	120
34	2,743	254.0	120
42	1,829	203.2	120
50	1,829	254.0	120
52	1,829	203.2	120
56	1,829	203.2	120
64	3,658	203.2	120
66	3,658	203.2	120
99	30.48	406.4	130
(b) Pipe	**L [m]**	**Rough [H-W]**	
NP_1	4,000	100	
NP_3	1,000	100	
NP_5	2,500	100	
NP_7	2,500	100	
NP_8	2,500	100	
NP_9	1,500	100	
NP_11	1,500	100	
NP_13	3,000	100	
NP_14	3,000	100	
NP_15	2,000	100	
NP_16	2,000	100	
NP_17	3,000	100	
NP_19	1,500	100	
NP_20	1,500	100	
NP_21	1,000	100	
NP_23	2,000	100	
NP_24	4,500	100	
NP_25	1,500	100	
NP_27	1,500	100	
NP_28	5,000	100	
NP_29	5,500	100	
NP_31	1,500	100	

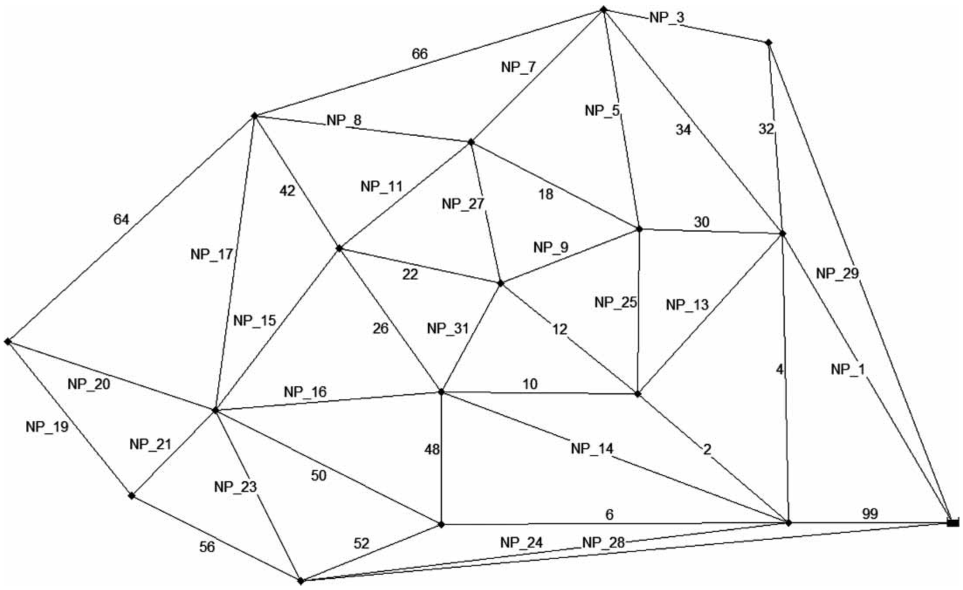

Figure 6 | Network AT-AC topology.

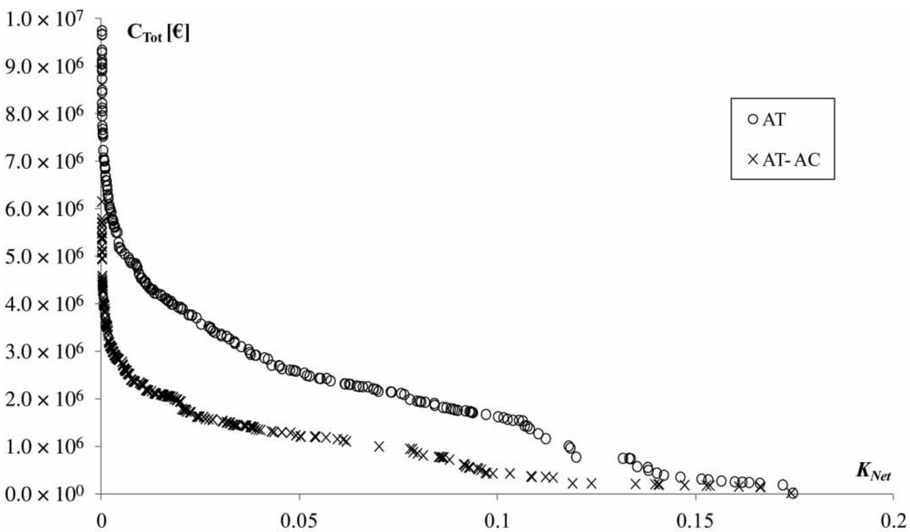

Figure 7 | Pareto fronts in comparison between classical and novel methodologies for Network AT.

expected, with an increase in costs. However, in this case the analysis was also undertaken with the novel methodology in which new pipes have been considered as decision variables. The results from this approach leads to solutions which dominate those of the classical redesign approach. Furthermore, examining the number of loops that are found to be optimal for different levels of risk, it can be seen that, for the AT-AC network, despite the potential for networks with larger numbers of loops to be generated, the maximum number identified

Table 8 | Cost analysis of the Pareto front solutions with the same network performance (a) and relative solutions for Network AT and Network AT-AC (b)

(a)

	K_{Net}	C_{Tot} [€]
A (3 loops), Classical methodology	0.086	1,820,000
AT-AC (3 loops), Novel methodology	0.085	777,655

(b)

AT (3 loops), Classical method	Pipe ID	2	6	10	12	18	22	26	48	4	30	
	DN	–	355	–	–	–	–	–	280	–	–	
	Pipe ID	32	34	42	50	52	56	64	66	99		
	DN	–	–	315	–	–	–	–	–	315		
AT-AC (3 loops), Novel method	Pipe ID	2	6	10	12	18	22	26	48	4	30	
	DN	–	–	–	–	–	–	closed	–	–	–	
	Pipe ID	32	34	42	50	52	56	64	66	99		
	DN	–	–	closed	–	–	–	closed	–	400		
	Pipe ID	NP_1	NP_3	NP_5	NP_7	NP_8	NP_9	NP_11	NP_13	NP_14	NP_15	NP_16
	DN	–	–	–	–	–	–	–	–	–	–	180
	Pipe ID	NP_17	NP_19	NP_20	NP_21	NP_23	NP_24	NP_25	NP_27	NP_28	NP_29	NP_31
	DN	–	–	280	180	–	–	–	–	–	–	–

as part of the optimisation is no greater than 3. This figure can thus be considered as a set of loops for the case study examined.

A comparison of optimal configurations, considering a same level of risk, obtained by means of the application of the two approaches is reported in Table 8 and Figure 8.

CONCLUSIONS

A WDS redesign optimisation problem has always been characterised by a trade-off between the performance of the system and the minimisation of cost. These objectives, however, could be greatly influenced by the existing network layout and its underlying level of redundancy.

In order to analyse the influence of the topological scheme of a network on the system redesign configurations, and thus on its performance, the classical approach used in the redesign techniques in which the network layout is considered as an input datum have been compared with a novel methodology in which the network topology has been considered as a decision variable in its own right and has been permitted free to be changed as would be the case in a design problem. The results were obtained by applying the multiobjective optimisation procedure based on a risk–cost methodology in which demand at network nodes has been modelled by means of a probabilistic approach. Based on the case study results obtained, this analysis has shown the noteworthy influence of the layout on the redesign solutions. An increase in the number of loops, through the addition of a small number of links, could lead to the realisation of significant savings in total redesign cost.

Moreover, from the comparison of the different topological configurations adopted it is possible to note that the increase in redundancy presents, nevertheless, a threshold looped configuration in terms of least-cost analysis. This result highlights the importance of the topological scheme on system performance that may have a greater influence on pipe diameter selection in network redesign. This could further lead to attempts to optimise the optimal redesign configuration, considering

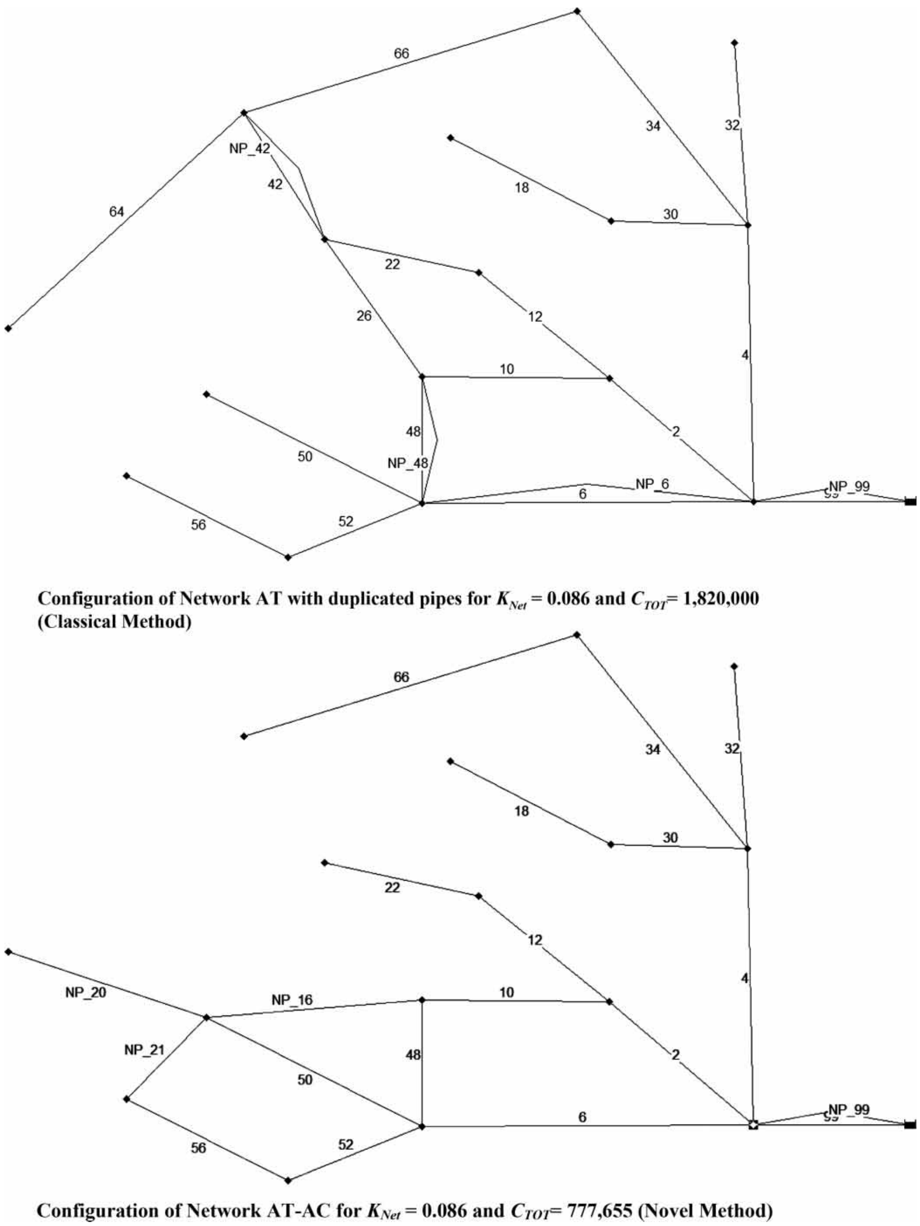

Figure 8 | Networks AT topologies for configurations presented in Table 8.

not just the duplication/substitution of existing pipes, but also the possibility of changing the original network layout.

The incidence of the system topology, indeed, is capable of driving the redesign optimal solution towards configurations that are both less costly and more reliable through greater redundancy.

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