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Seismic reliability assessment of water distribution networks

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Abstract

A methodology for assessing the seismic risk in a WDN is presented here as obtained from the analysis of network mechanical reliability when pipe failures are due to earthquakes. The mechanical reliability analysis is performed considering WDN topological changes subsequent to multiple failure scenarios; while pipe failure probability law is brought from the American Lifeline Alliance that assumes empirical vulnerability functions derived from data collected during recent earthquakes in the Pacific area. A multi-objective optimization is run to explore the space of contemporary pipe failures which may occur during an earthquake. The method is applied and discussed using C-Town network.

Keywords: Seismic Reliability Analysis; Unsupplied water; Vulnerability functions; Multi-objective optimization.

1. Introduction

Water Distribution Networks (WDNs) as well as transport, telecommunications and electricity networks can be defined as lifelines. Their damaging as a result of an earthquake could lead to dramatic scenarios comparable to those directly caused by the event itself.

The seismic design/analysis of lifelines can be dated back to the work of Katayama et al. (1975), which dealt with seismic design of buried oil pipelines, while for the water supply systems the first scientific work in this direction was from O'Rourke (1988). The seismic design/verification of buried pipelines is very difficult indeed, since they spread over a wide area of the urban territory of which, in most cases, few detailed knowledge of geological and geotechnical features are available. Nonetheless, such soil characteristics are the key elements for

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the occurrence of damage on individual mains, together with the magnitude of the earthquake. This have led water utilities to focus on how to reduce earthquake effects on existing water systems, in terms of physical damage mitigation (strengthening the different components of the system with the aim of reducing the likelihood of damage) and in terms of lessening of consequences (implementing emergency management plans and repair of the essential parts of the system). In both cases, the Authorities need to know in advance the vulnerability of a WDN by estimating the seismic risk associated with the probability of failures and consequent damages, ALA (2001). To this aim, it is important to search for the contemporary events caused by an earthquake, which impair WDN reliability.

For WDNs, reliability can be defined as the capacity to deliver water to costumers with adequate levels of service (flow and pressure) also under abnormal operating conditions, Xu and Goulter (1999). When these abnormal operating conditions are caused by changes of network topology due to the interruption of the operation of some network elements (e.g. pipes, valves, pumps, etc.) the relevant analysis is called mechanical reliability assessment. Several measures and strategies have been developed so far with respect to mechanical reliability, e.g., Xu and Goulter (1999); Farmani et al. (2005), nonetheless none of them investigated the most critical scenarios caused by multiple failures, also considering the existing isolation valves system which needs to be closed in order to isolate the failed pipes. In fact, even considering a network of limited size, there are a huge number of possible combinations of failure.

This paper presents a methodology to perform a WDN mechanical reliability assessment considering multiple failures of network elements, assuming different failure probabilities for each pipe, in order to reflect a realistic earthquake scenario. The methodology presented herein considers only pipes as the WDN elements to be damaged by the earthquake. Future works will include also vulnerability functions for other elements such as tanks, reservoirs, pumps and valves.

The key points of the proposed methodology are:

- evaluation of the probability of failure for each pipe, as consequence of the assumed earthquake: to this aim, the failure probability of the single pipe was determined following the provisions of the American Lifeline Alliance, ALA (2001), by means of the vulnerability function, which is an empirical expression determined from several databases recorded after earthquakes in Japan and USA;
- searching for the most destructive scenarios for a WDN (combination of contemporary failure events), using the multi-objective optimization to search the minimum number of damaged pipes that would result into the minimum supplied water demands. The optimization problem was solved by means of the WDNetXL system, Giustolisi et al. (2011);
- calculate the seismic risk of each critical scenario based on its joint probability to occur, and the consequent damages (i.e. unsupplied water demand).

The method was applied to C-Town supply network, a real WDN detailed in Ostfeld et al. (2012).

2. Used models for seismic vulnerability

To implement measures for mitigation of earthquake consequences, water utilities need methods to estimate the probability of failure caused by seismic loads, thus identifying the most vulnerable WDN elements or districts. To this end, researchers have produced some relationships named vulnerability functions (or seismic fragility formulations) expressing the Rate of Repair (RR) per unit length of buried pipes, as a function of seismic shaking, measured in terms of Peak Ground Velocity (PGV) or Permanent Ground Deformation (PGD). Taking into consideration of the PGV means to account for propagation of seismic waves. These generate transient deformations of the ground (Transient Ground Deformation - TGD) involving a very large area and therefore most of the underground pipes, sometimes causing their failure (e.g., slippage, bending moments, shear stress, etc.). Conversely, the analysis of PGD requires in-depth geotechnical knowledge of the site, which is rarely available, ALA (2001). Therefore, in what follows, all the relationships used will be referred to the vulnerability functions including the PGV.

In general, the *RR* of a pipe depends on several factors, such as the material *M*, the joint type *G*, soil corrosivity *C*, pipe diameter *D* and the *PGV*. Thus, the functional relationship that expresses these dependencies is:

$$RR = f\left(M, G, C, D, PGV\right) \tag{1}$$

However, these data are rarely available, as are difficult to measure after (or during) an earthquake. Therefore, researchers have tried to define *f* (•) on the basis of empirical evidence from integrated engineering judgments and, sometimes, using analytical formulations. Essentially, the construction of the vulnerability functions is based on the availability of a database containing information on kilometers of pipes affected by a seismic event and their repair rates after the earthquake. Then, this approach resulted into few empirical data available, very heterogeneous in nature, which are therefore difficult to generalize, but are specific for each individual earthquake and/or network analyzed.

Among the first works in this direction, Eidinger (1998) presented the vulnerability functions developed from data recorded on more than 3300 miles of pipes that were subjected to different levels of shaking during the Loma Prieta Earthquake (USA - 1989), differentiated according to the material (welded steel, cast iron, asbestos cement). More recently, in 2001, the American Lifeline Alliance starting from existing studies, including those just mentioned, and on the basis of their own elaborations defined a mathematical structure for the function of vulnerability, as shown in Eq. (2). On the one hand, this general pattern is very simple and is suggested when no specific data on pipelines are available (e.g., material, joint, refill, etc.). On the other hand, a modified function of vulnerability, as in Eq. (3) was proposed when more information on the pipeline asset are available

$$RR = a \cdot PGV^{b} \tag{2}$$

$$RR = K \cdot a \cdot PGV^{b} \tag{3}$$

where the rate of repair *RR* is indicated per unit length (1000 feet of pipe) and the *PGV* is expressed in in/s. *a* and *b* are coefficients obtained by linear regression from the available database; *K* is a coefficient to be used for the specific condition of the pipe (e.g., material, joint type, soil, diameter, etc.).

Table 1. Coefficients *K* for the function of vulnerability, according to Eq. (3), see ALA (2001)

Pipe material	Joint type	Soils	Diameter	K
Cast iron	Cement	All	Small	1.0
Cast iron	Cement	Corrosive	Small	1.4
Cast iron	Cement	Non-Corrosive	Small	0.7
Cast iron	Rubber gasket	All	Small	0.8
Welded steel	Lap- Arc welded	All	Small	0.6
Welded steel	Lap- Arc welded	Corrosive	Small	0.9
Welded steel	Lap- Arc welded	Non-Corrosive	Small	0.3
Welded steel	Lap- Arc welded	All	Large	0.15
Welded steel	Rubber gasket	All	Small	0.7
Welded steel	Screwed	All	Small	1.3
Welded steel	Riveted	All	Small	1.3
Asbestos cement	Rubber gasket	All	Small	0.5
Asbestos cement	Cement	All	Small	1.0
Concrete w/Stl Cyl.	Lap- Arc welded	All	Large	0.7
Concrete w/Stl Cyl.	Cement	All	Large	1.0
Concrete w/Stl Cyl.	Rubber gasket	All	Large	0.8
PVC	Rubber gasket	All	Small	0.5
Ductile iron	Rubber gasket	All	Small	0.5

Based on data collected in 12 different earthquakes, for a total amount of 81 data points, the ALA has proposed its formulations of the vulnerability functions. The most common material in the database is cast iron (38 data points) followed by steel (13), asbestos cement (10), ductile iron (9), and concrete (2). Other 9 data points represent both cast and ductile iron pipe combined. In terms of pipe diameter, the database contains mostly those sizes associated with distribution main systems; only 8 data points were identified as specifically for large-diameter pipe greater than 12 inches. In Eq. (2) and Eq. (3), the coefficients b = 1 and a = 0.00187 is the median slope of the data point set in a diagram having on x-axis the PGV and on the y-axis the RR. This means that the line defined by this model has the property of having equal numbers of points above and below it. It is worth to note that ALA elaborated further simple models (e.g., two-parameter linear model, logarithmic model), but both models are almost equivalent, especially when considering the scatter in the data points. More details can be found in ALA (2001).

ALA (2001) reports also additional analyses performed to assess the influence of pipe material, diameter and earthquake magnitude. For different pipe materials, relative vulnerability was explored by computing linear models for each material and taking the ratios of the slope coefficients (parameter a). This leads to the determination of K to be used in Eq. (3) for different combinations, as reported in Table 1, together with the recommendations proposed by the ALA. In Table 1, small diameters are from 4 to 12 inches, while large diameters means >12 inches. The coefficients in Table 1 are for pipelines which do not account for seismic design specific to the local geologic conditions.

3. Single pipe failure algorithm

In this work the probability of failure for the single pipe will be determined according to the indications of the ALA (2001). Thus, given the j^{th} pipe of length L, its probability to fail due to an earthquake (P_j^{out}) can be written as a function of several factors, related to asset and external stresses, as:

$$P_j^{out} = g\left(L, M, G, C, D, PGV\right) \tag{4}$$

with reference to the symbols in Eq. (1). In ALA (2001) the function $g(\bullet)$ is defined on the basis of the Poisson probability distribution

$$P\left(x = y\right) = \left(\lambda L\right)^y \cdot e^{-\lambda L/y!} \tag{5}$$

where x is a random variable denoting the number of times the event of a broken pipe occurs, λ is the breakage rate (RR in this case), and λL is the average number of events occurring over length L of the pipe. The Eq. (5) means that the assessment of the probability of failure $P(x = y)$ requires the knowledge of the number of times (y) the break occurs for the j^{th} pipe. The problem is solved by determining the probability of service of the pipe ($P_j^{service}$), which is obtained from Eq. (5) by assuming breaks (y = 0), thus having

$$P_j^{service} = \left(\lambda L\right)^0 \cdot e^{-\lambda L/0!} = e^{-\lambda L} \tag{6}$$

and consequently the probability of failure of the j^{th} pipe of length L is equal to

$$P_j^{out} = 1 - e^{-\lambda L} \tag{7}$$

where λ = RR can be calculated using Eq. (2) or Eq. (3) with the coefficients K in Table 1. The implicit assumption of using Eq. (7) is that the probability of failure of the j^{th} pipe increases with its length, which is technically sound also considering the physical phenomena involved in such analysis.

4. Reliability analysis paradigm

The analysis of WDN reliability requires the hydraulic simulation of the system under variable boundary conditions (i.e. topology in case of mechanical failures). For this reason, the WDN model used herein includes: (i) pressure-driven modeling of all components of water demand, Giustolisi and Walski (2012) including background leakages, Giustolisi et al. (2008); (ii) the presence of directional devices and pressure control valves, Giustolisi et al. (2012); (iii) the automatic detection of current network topology in order to reproduce both intended and unintended segment isolations due to closing gate valves and the effects of automatic closing of control/directional devices. The used hydraulic model is reported in matrix form in Eqs. (8); such equations assume that actual demands (i.e. water outflows) at each node ($\mathbf{d}_n$) depend on relevant head according to pressure-driven modeling assumption

$$\begin{cases} \mathbf{A}_{pp}\mathbf{Q}_p + \mathbf{A}_{pn}\mathbf{H}_n & = & -\mathbf{A}_{p0}\mathbf{H}_0 + \mathbf{H}_p^{pump} \\ \mathbf{A}_{np}\mathbf{Q}_p - \mathbf{d}_n\left(\mathbf{H}_n\right) & = & \mathbf{0}_n \end{cases} \tag{8}$$

The hydraulic status of the WDN is represented by nodal heads ($\mathbf{H}_n$) and pipe flow rates ($\mathbf{Q}_p$); the current network topology is defined by the topological incidence sub-matrices $\mathbf{A}_{pn}=\mathbf{A}_{np}^{\mathrm{T}}$ and $\mathbf{A}_{p0}$; the vector $\mathbf{d}_n(\mathbf{H}_n)$ contains water demands lumped in the nodes. The used hydraulic simulation model can account also for the existence of the Isolation Valve System (IVS) which can determines segments in the analyzed network. The automatic detection of current network topology due to valve shutdowns in this work is based on the methodology reported in Giustolisi and Savic (2010). Based on the existing IVS, this methodology permits to detect those portions of the network which are still connected to water sources when one or more segments are detached by closing relevant gate valves.

An IVS entailing the so called N-valve rule (Walski, 1993) permits to detach each single pipe from the network by closing two isolation valves put at its ends; while the IVS in real WDNs, which are far from entailing the N-valve rule, usually requires the detachment of all elements falling into the same segment in order to isolate just one pipe. This, in turn, might cause significant alteration of the system due to mechanical failures.

The subsequent changes of network topology (i.e., matrices $\mathbf{A}_{pn}$ and $\mathbf{A}_{p0}$ in model (8)) are obtained by eliminating all isolated segments and unintentionally disconnected pipes, before the simulation run.

4.1. Assessment of WDN reliability under multiple mechanical failures

Although the WDN model in Eq. (8) permits the simulation of all single-failure scenarios in a reasonable computational time for large real WDNs with a near N-valve rule IVS, the number of possible combinations of *k* simultaneous failures is dramatically higher than the number of single-failure events, even for small networks. This makes infeasible the exhaustive simulation of all possible scenarios and motivates the proposal for the approach reported herein.

From water supply service perspective, the worst scenarios are those resulting into the lowest supplied demand as a consequence of the minimal number of failures. Nonetheless, it is possible that different combinations of simultaneous failures result into severely disruptive scenarios. Thus the problem is not only to find one worst combination of *k* simultaneous events but, rather, a set of failures scenarios (i.e. the failure of *k* pipes) resulting into severe WDN malfunctioning, although they entail very different topological and hydraulic WDN states. In fact, these scenarios are of technical interest in supporting the planning of risk mitigation works.

In addition, achieving a set of *k*-failures scenarios, for any *k,* makes the procedure more robust in face of all uncertainties surrounding the boundary conditions assumed to compute the global WDN performance (e.g. the total supplied demand) as, for example the uncertainty on human water requests.

The strategy proposed herein is based on the solution of a multi-objective optimization problem where two objective functions are simultaneously minimized: (i) the number of simultaneous events (i.e. leading to segment(s) detachment); (ii) the demand supplied to customers, as predicted by the WDN simulation model. Such objectives are conflicting since it can be reasonably assumed that WDN hydraulic performance worsens as the number of simultaneous failure events increases.

Each pipe in the WDN is assigned with a different probability of failure as response to an assumed earthquake, according to Eq. (7). The searching paradigm is based on Genetic algorithms (GA); in particular, the OTPImized Multi Objective GA (OPTIMOGA), Laucelli and Giustolisi (2011), is used here. The evaluation of fitness of each solution at generation is performed in OPTIMOGA by assigning it a rank according to the methodology proposed by Fonseca and Fleming (1993). The OPTIMOGA based procedure permits also to set the maximum allowable rank r_{max} (= 8 in the case study) so that the final set of failure combinations entails the highly disruptive scenarios which are meaningful from a WDN management perspective, Ugarelli et al. (2012). This means that, for each number of simultaneous k events (with k = 1,…, k_{max}) the procedure returns up to 8 failure scenarios (even representing very different WDN topological and hydraulic states) which are all technically meaningful in determining the WDN seismic risk. It is worth to observe that the search is upper bounded to k_{max} simultaneous events equal to the minimum number of failures which simultaneously detach water sources (e.g. reservoirs, tanks), and thus is known a priori from the WDN layout.

Therefore, each solution on the Pareto front, i.e. a number of contemporary failure events, reflects:

- the percentage of the unsupplied demand at WDN nodes, which can results from the detachment of some pipes in the network, even more than those really failed. This is why the simultaneous failure of a number of pipes can cause the isolation from the water source(s) of a network segment (district) according to the available IVS, sometimes incurring in unintentional disconnection of some nodes and pipes;
- the joint probability of failure for the relevant pipes, which can be written as

$$P_s^{out}\left(A,B,C\right)=P_j^{out}\left(A\right)\cdot P_j^{out}\left(B\right)\cdot P_j^{out}\left(C\right) \tag{9}$$

in the simple case of three mutually independent failure events A, B and C, Benjamin and Cornell (1970), given that for each pipe the $P_j^{out}(\bullet)$ is that in Eq. (7).

In the end, for each solution on the Pareto front, the seismic risk can be written as

$$R_s = P_s^{out}\cdot d^{uns} \tag{10}$$

where d^{uns} is the damage (% unsupplied water) caused by the simultaneous disruption of a number of pipes, and P_s^{out} is the joint probability of failure as consequence of an earthquake.

5. Case of study

In this work, the reliability analysis is applied to a real network, C-Town, Ostfeld et al. (2012). The network topology as extracted from the C-Town geographical information system is described in Fig. 1. It comprises 444 pipes and 396 nodes, with 1 water sources (i.e. reservoir R1 in Fig.1), 7 tank and 5 pumping stations (a total amount of 11 pumps). The C-Town network is divided in 5 district meter areas (DMAs) each one having at least two pumps providing the right pressure and one or two cylindrical tanks with diameters and minimum and maximum levels as estimated from the municipality master plan document. The IVS has been assumed to comprise 1 valve each serial node, for a total amount of 888 valves; additionally, due to elevation changes there are a few pressure reducing valves (PRV) in the system. The pressure to correctly supply water to customers is 20 m. Background leakages are not accounted for in this study.

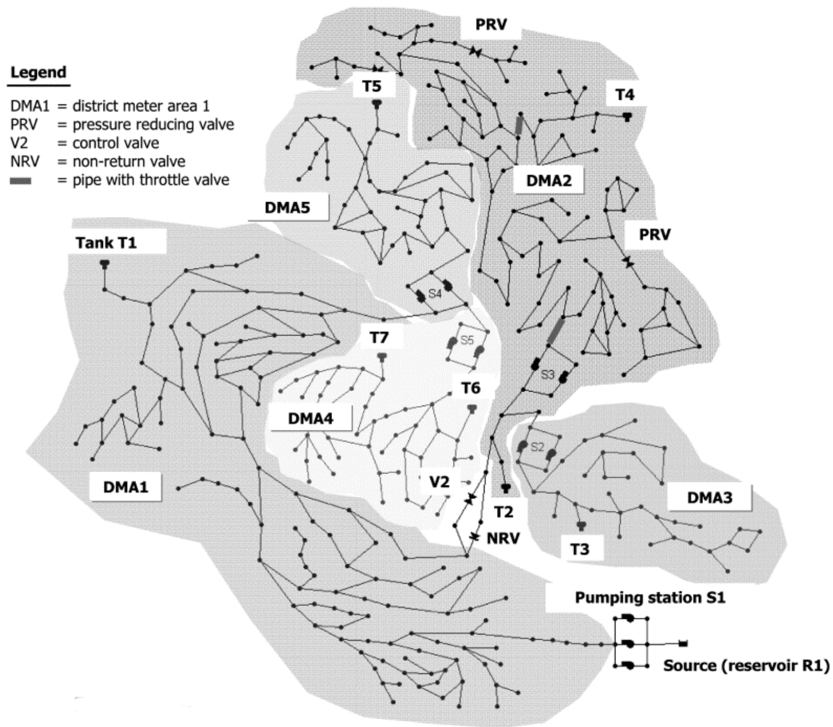

Fig. 1. C-Town layout (source Ostfeld et al., 2012).

The use of Eq. (3) requires the knowledge of certain characteristics of pipes, which have been here assumed as working hypotheses, as listed in Table 2, according to Table 1. Note that for both the assumed configurations all soil types are considered. As working hypothesis, the highest PGV values in the database used by the ALA were here assumed, ranging from 33.6 to 55.1 in/s.

Table 2 – Asset features of pipes assumed for the study WDN (working hypothesis).

Diameter	Material	Diameter	*K*
Small	Cast iron	Rubber gasket	1
Large	Concrete w/Stl. Cyl.	Cement	0.8

In particular, according to the characteristics of the available database, the small diameters in this WDN was assigned to the PGV values relevant to cast iron (i.e. considering groups based on proximity) and large diameters was assigned to the PGV values relevant to concrete and asbestos cement. Finally, pumps have been assigned with a low failure probability, since it is expected that they are installed in safer conditions than buried pipes, and considering that a dedicated vulnerability function has not been used in this work. All analyses have been performed by using the WDNetXL system, Giustolisi et al. (2011).

6. Results and discussion

The described procedure for the seismic reliability assessment of the C-Town network returned 104 worst failure scenarios. For the sake of brevity, Table 3 reports only the scenarios with rank 1, 2, 3 and 4 for all the possible number (k) events, featured by the number of failure events, the percentage of unsupplied water, the joint probability and the seismic risk calculated using Eq. (10). It is evident that detaching the 8 water sources (T1, T2, T3, T4, T5, T6, T7 and R1) from the C-Town network prevents any water to be supplied; such worst scenario requires the detachment of 8 segments joining water sources to the network (i.e., k_{max} = 8) since the reservoir and the tanks are connected to the WDN by one segment each. Vice versa, when all network segments are normally connected, 100% of requested demand is correctly supplied. These two extreme situations are indicated with a triangle and a square in Fig. 2, where the worst failure scenarios listed in Table 3 are plotted. In Fig. 2 the x-axis shows the number of pipes contemporarily failed, and the y-axis shows the supplied water demand (as percentage of the required demand), varying from 0 (the case with the whole WDN disconnected) to 100 (the case with the network fully connected to water sources).

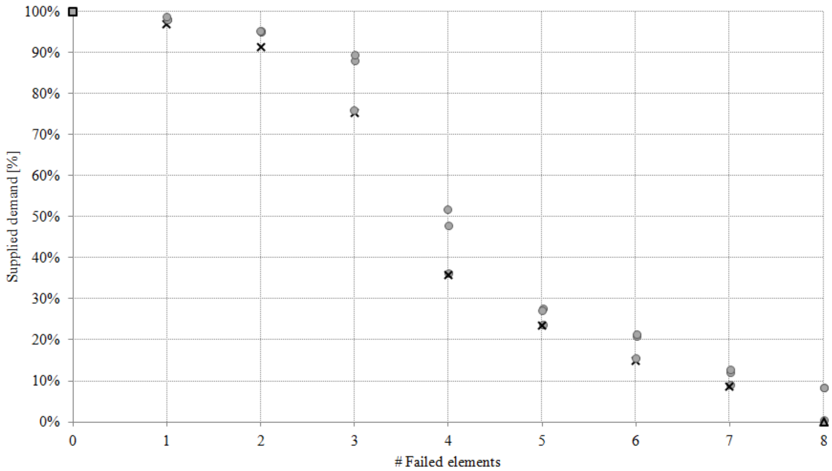

Fig. 2. Worst pipe failure scenarios for the C-Town network.

Fig. 2 shows also the worst scenarios having 1 to 7 contemporary failures (cross points), and some scenarios with a number of 8 to 1 simultaneous failures that, although not the most critical, can cause severe reduction in the supplied water.

A general consideration can be done looking at values of unsupplied demands (damage of failure scenarios): the particular layout of the C-Town network allows limiting damages of a possible earthquake up to 3 simultaneous pipe interruptions. In fact, considering the solution number 10 shadowed in Table 3, the worst scenario (rank 1) for 3 concurrent failures, the presence of tanks and/or water source allow delivering the 75% of requested water demand to customers. Conversely, when considering the worst scenario (rank 1) for 4 concurrent failures, e.g. solution number 14 shadowed in Table 3, more problems arise to satisfy all water demands in the network, and the relevant damage consists of 64% un unsupplied demands. Additionally, in this last case, the joint probability, and consequently the seismic risk, is of two orders of magnitude higher than solution number 10, indicating a more critical situation to consider for water managers.

From technical perspective, the original layout of this network permits to prove the hydraulic consistency of results since the most critical failure scenarios are those which progressively isolate the DMAs from relevant local source (tanks).

Finally, it is to note that solutions with a higher number of simultaneous failures, involving the failure of pipes connecting the network to the tanks, presents an unsupplied demand equal to 100% but a very low risk compared to all others, due to the very low joint probability of such event.

Table 3. The most critical failure scenarios for the C-Town.

Solution #	Number of Events	Unsupplied water	Joint Probability	Risk
1	0	0.00%		
2	1	3.05%	2.37E-02	7.23E-04
3	1	1.77%	6.01E-02	1.06E-03
4	1	1.69%	4.81E-02	8.12E-04
5	1	1.35%	1.03E-01	1.39E-03
6	2	8.63%	1.39E-04	1.20E-05
7	2	4.82%	1.42E-03	6.87E-05
8	2	4.74%	1.14E-03	5.40E-05
9	2	4.40%	2.44E-03	1.07E-04
10	3	24.59%	9.07E-09	2.23E-09
11	3	23.99%	1.04E-09	2.50E-10
12	3	11.68%	3.30E-06	3.85E-07
13	3	10.40%	8.35E-06	8.69E-07
14	4	64.18%	1.02E-06	6.56E-07
15	4	63.59%	1.17E-07	7.46E-08
16	4	51.99%	1.24E-07	6.46E-08
17	4	48.22%	3.34E-07	1.61E-07
18	5	76.58%	1.10E-08	8.44E-09
19	5	75.98%	1.27E-09	9.62E-10
20	5	72.81%	2.96E-08	2.16E-08
21	5	72.21%	3.40E-09	2.46E-09
22	6	85.21%	3.20E-10	2.73E-10
23	6	84.61%	3.67E-11	3.11E-11
24	6	78.97%	3.70E-10	2.92E-10
25	6	78.38%	4.25E-11	3.33E-11
26	7	91.37%	3.99E-12	3.65E-12
27	7	90.78%	4.58E-13	4.16E-13
28	7	87.60%	1.07E-11	9.41E-12
29	7	87.01%	1.23E-12	1.07E-12
30	8	100.00%	1.16E-13	1.16E-13
31	8	99.40%	1.33E-14	1.32E-14
32	8	91.37%	4.11E-13	3.76E-13
33	8	91.37%	2.85E-13	2.61E-13

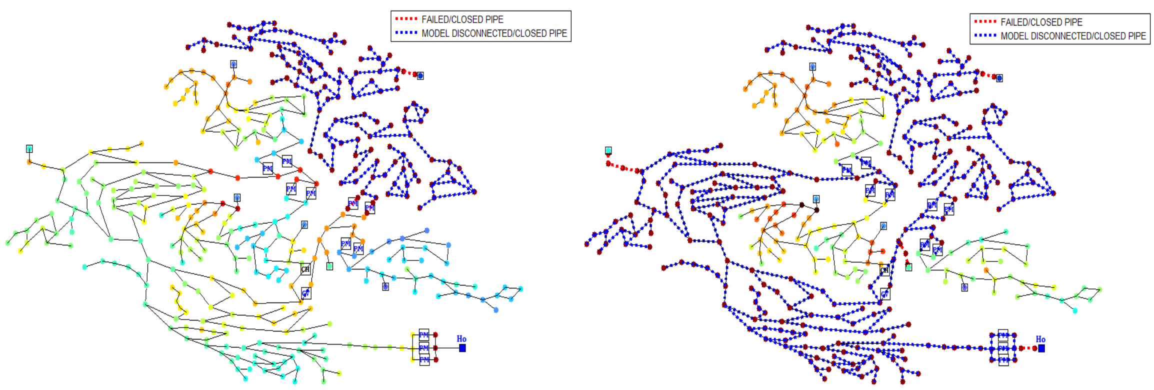

Fig. 3. Scenario number 10 (left) and Scenario number 14 (right).

7. Conclusions

A new methodology to assess seismic reliability of a WDN has been proposed, based on a probability density function for pipe failure due to an earthquake indicated by the ALA. Such relationship includes a vulnerability function, defined using observed values of the *PGV* (Peak Ground Velocity) during many earthquakes in Japan and California between 1971 and 1995.

The strategy to identify the most disruptive combinations of failure events exploits a multi-objective genetic algorithm paradigm (i.e. OPTIMOGA). Results show that the seismic risk for a water distribution network depends on the probability of breaks of individual pipes and by the isolation valves system of the network (which are assumed here to work properly and activate in emergency conditions).

The methodology has the potential to be a valuable tool to support decisions of water utility to improve WDN preparedness to earthquakes, since it is able to indicate the most critical scenarios in terms of possible alterations of topology and consequent unsupplied demand to customers. Additionally, it can support managers to effectively allocate financial resources for risk mitigation work and to prepare appropriate emergency plans focused at maintaining the functionality of the lifeline "aqueducts". Future works will focus on the enhancement of the used vulnerability functions and inclusion of vulnerability functions for other WDN elements such as tanks, reservoirs, pumps and valves.

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