SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

VUNLNERABILITY ASSESSMENT OF WATER DISTRIBUTION NETWORKS USING SURVIVAL ANALYSIS

Michalis FRAGIADAKIS[1] and Symeon CHRISTODOULOU[2]

ABSTRACT

We discuss the vulnerability assessment of water supply networks combining data of past nonseismic damage and the seismic vulnerability of the network components. Historical data are obtained using records of damage (pipe breaks) that occur on a daily basis throughout the network and are processed to produce survival curves. The fragility of the network components is assessed using the approach suggested in the ALA guidelines. The network reliability is assessed using Graph Theory and Monte Carlo simulation. The proposed reliabiity-assessment method is demonstrated on a district metered area of the water network of Limassol, Cyprus. This approach allows the estimation of the probability that the network fails to provide the desired level of service and allows the prioritization of retrofit interventions and of capacity-upgrade actions pertaining to existing water pipe networks.

INTRODUCTION

The work presented herein attempts to provide a methodology for a system-wide analysis based on component analysis, network topology and, most importantly, survival analysis in order to include the effects of a network's past performance on its seismic reliability assessment. This paper combines data on historical non-seismic performance of urban water distribution networks (UWDN) and their components by use of survival analysis. The goal is to calculate the seismic reliability of a network combing daily performance measurements with the approach suggested by the guidelines of the American Lifeline Alliance (ALA 2001) for the seismic assessment of the network. The ALA guidelines present procedures and pipe fragility relationships that can be used to evaluate the probability of earthquake damage to water transmission systems and to make informed decisions on how to mitigate risks. However, the generic form of the pipe fragility curves obtained through the ALA, and other, methods does not take into consideration a network's past performance and its effects when calculating the pipe repair rates due to seismic loading.

The reliability of a water pipe network can be calculated if the vulnerability (also termed fragility) of every element of the water network is known. Although water pipe networks consist of several elements (pipes, house connections, tanks, pumps, etc.), focus is given on the pipes, which are, not only the most important component in a piping network but they are also the most difficult component to inspect and replace. Many possible risk-of-failure parameters can be identified (Romero *et al.* 2010). Our methodology takes into consideration the fragility that corresponds to pipe failures that occur frequently during the everyday operation of the water network and also more severe, but less often, failures due to earthquakes. The pipe vulnerability due to nonseismic causes is assessed using survival analysis techniques on available everyday measurements. Survival analysis considers a number of parameters, e.g. number of observed previous breaks (NOPB), pipe material, diameter or

[1] Lecturer, National Technical University of Athens, Athens, mfrag@mail.ntua.gr
[2] Ascosciate Professor, University of Cyprus, Nicosia, schristo@ucy.ac.cy

age that affect the pipe survival curves (Christodoulou 2011) in order to develop survival/hazard rates and time-to-failure curves for system components based on a multitude of risk-of-failure factors and data stratifications. To account for the vulnerability due to seismic hazard, we propose a rational approach for combining the pipe seismic fragilities with the results of survival analysis in order to consider the effect of previously observed breaks in the network.

Even though the procedure proposed herein is presented based on performance data from urban water networks of the island of Cyprus, it is general in scope and applicable to any locale with historical records of pipe-break incidents in its water distribution network. Being a South European island, Cyprus has suffered during the last years from low rainfalls and shortage in its water reserves. Under such conditions, a common practice followed by water distribution agencies has been to periodically interrupt the water flow in different areas of the city network for variable time intervals, e.g. 12 hours of water supply every 48 hours. This practice offers a more rational treatment of the water resources, but is also considered responsible for increasing the failure rate of the network pipes. The worsening failure rate in the water pipe networks of all major cities of the island prompted the initiation of an extensive program of monitoring and keeping track of the every damage incident, in order to be able to assess the network conditions and assist its proper maintenance. The post-processing of the vast amount of available data is performed using survival analysis tools and producing pipe survival curves that allow considering the effect of different parameters (e.g. material, age, diameter) on the failure rate. Our resukts indicate that, although the island is located in a moderate seismicity environment, the seismic vulnerability of its water distribution networks increases to considerable levels due to the deterioration of the pipe properties.

SYRVIVAL ANALYSIS

Survival analysis is a branch of statistics dealing with deterioration and failure over time and involves the modeling of the elapsed time between an initiating event and a terminal event (Hintze 2001). In the case of piping networks, such initiating events can be the installation of a pipe, a water-leak observation or the start of a pipe treatment. Cases of terminal events can be a relapse of a previous leak, a fix or a failure. The method is based on estimating the reliability of a system network and its lifetime subject to multiple risk factors. The aim is to provide answers on the population fraction (pipes) that survives past an expected lifetime, on the effect of the various risk factors on the systems lifetime, and on the probability of survival and the expected mean time to failure (Hintze 2001, Hosmer *et al.* 2008). The data values used in the analysis are a mixture of both complete and censored observations. In the former case, a terminal event is thought to have occurred, whereas in the latter case, a terminal event has not occurred. A terminal event is assumed to occur just once for every subject.

A pipe's survival function, S for elapsed time, T until the occurrence of a pipe failure is given by the expression:

$$S(t) = \int_T^{\infty} p(x)dx = 1 - P(t) \tag{1}$$

Thus, the survival function is the probability that the time to failure is longer than some specified time t. Moreover, $P(t)$ is the cumulative distribution function that denotes the probability that a pipe survives until time t and $p(t)$ is the corresponding probability density function. According to Equations (2) and (3), the rate of the survival function is denoted as $h(t)$ and provides the probability that a pipe at time T experiences the event in the next time instant. The cumulative hazard function $H(T)$ is the integral of $h(t)$ from 0 to T, and therefore,

$$S(t) = \exp\left[-\int_0^T h(x)dx\right] = \exp\left[-H(T)\right] \tag{2}$$

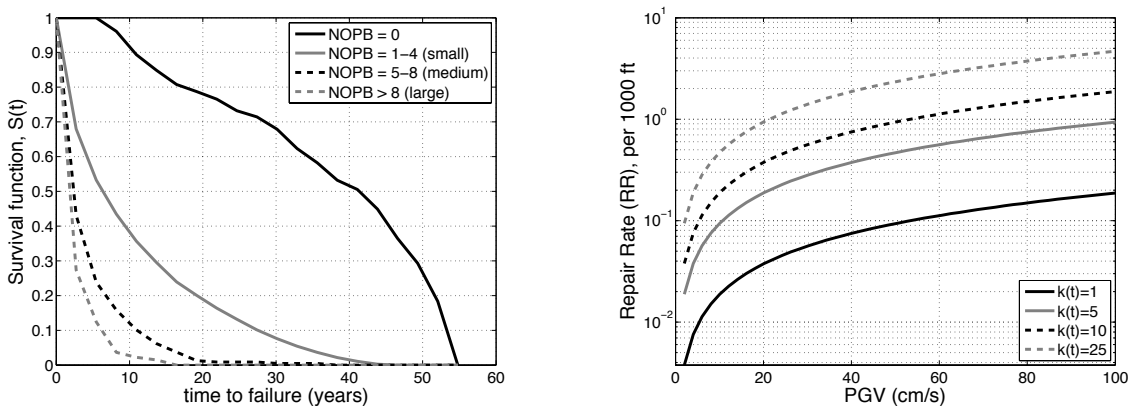
and

$$h(T) = p(T)/S(T) \tag{3}$$

The survival function S is usually the primary quantity of interest and is numerically calculated using kernels, such as the Epanechnikov kernel and the Kaplan–Meier estimator (Kaplan and Meier 1958). The Kaplan–Meier estimator is of particular importance because it is non-parametric and therefore, relies on data rather than analytical equations and probability density functions in order to produce the survival curves. A plot of the Kaplan–Meier estimate of the survival function is a series of horizontal steps of declining magnitude, which approaches the true survival function for the population in study and whose values between successive distinct sampled observations are assumed to be constant. Another important advantage of the Kaplan–Meier curve is that the method can take into account both left and right-censored data. When no truncation or censoring occurs, the Kaplan–Meier curve is equivalent to the empirical distribution function.

In terms of piping networks, the survival function has been shown to be dependent on several factors, the most important of which are the "number of previous breaks" (NOPB), the age and the material of the pipes (Christodoulou and Deligianni 2010). These risk factors have been studied extensively both when acting separately or together. The nonparametric survival analysis produces the effects of such risk-of-failure actions on the network, clustered by risk factor and its subgroups, and enables us a deeper insight into the behavior of the piping network. For example, a survival analysis reported by Christodoulou and Ellinas (2010) of an urban WDN under abnormal operating conditions, revealed almost identical survival curves for the network mains and its house connections, but when clustered by the NOPB, the survival curves varied substantially.

A typical set of survival curves is shown in Figure 1a. The curves have been derived from real data, offered to us by the Water Board of Limassol (Cyprus), and refer to asbestos cement (AC) pipes. The data have been clustered according to the NOPB, and four survival curves are derived. The four curves correspond to 0, 1-4, 5-8 and more than 8 previous breaks and are denoted as 'zero', 'small', 'medium' and 'large' NOPB clusters, respectivelly. According to Figure 1, even in the 'NOPB=0' case, the pipe will have to be eventually replaced after approximately 55 years, whereas a pipe that has already broken more than eight times is not expected to survive more than 18 years. Moreover, a pipe that has broken at least once, is considerably more vulnerable compared to an intact pipe that has never been damaged.



(a) (b)

Figure 1 (a) Typical survival curves; (b) The effect of k on the repair rate (RR) as function of the peak ground velocity (PGV).

SEISMIC VULNERABILITY ANALYSIS OF WATER PIPES

The seismic vulnerability (or fragility) of buried pipelines is discussed in the ALA document (ALA 2001). The ALA document provides vulnerability curves for water pipes using observations from past disruptive earthquakes. The failure parameters that affect buried pipes are identified and vulnerability functions are proposed. The vulnerability functions are defined as functions of the peak ground velocity (PGV) and the permanent ground deformation (PGD). PGV is related with strong ground

shaking caused by seismic wave propagation, whereas PGD is used to measure ground failure factors that include landslides, liquefaction, ground settlement and fault crossing. Parameters that also affect the vulnerability of a pipe are also the diameter, the age, the year of construction and possible discontinuities along the pipe. The pipe vulnerability functions of the ALA document, provide the repair rate (*RR*) per 1000ft of pipe length and have the form:

$$\begin{aligned} RR_{PGV} &= K_1 \cdot a \cdot PGV \\ RR_{PGD} &= K_2 \cdot b \cdot PGD^c \end{aligned} \tag{4}$$

when the units for PGV and PGD are in inch(es) per second and inch(es), the constants *a*, *b* and *c* are equal to 0.00187, 1.06 and 0.319, respectively. If SI units are preferred, PGV and PGD are expressed in meter(s) per second and meter(s), and the constants are equal to 0.001425, 4.281 and 0.319, respectivelly. Tabulated values are provided for K_1 and K_2 depending on the material of the pipe. K_1=K_2=1 refers to pipes made from cast iron or asbestos cement. The pipe *RR*s of Equation (4) can be due to a complete fracture, a leak or a damage to an appurtenance of the pipe, or any other reason that requires the water agency to intervene. For typical water pipe networks, a rule of thumb is that for failure due to wave propagation, 15–20% of failures are breaks and the rest are leaks, whereas for failures due to PGD, 80–85% are breaks that result to the loss of pipeline hydraulic continuity (Ballantyne 1990).

Once the *RR* is known, that is, the number of leaks/breaks per pipe length, the failure probability of the pipe can be easily calculated. The failure probability of a pipe is equal to one minus the probability of zero breaks along the pipe. Using the well-known exponential distribution CDF formula, the pipe failure probability P_f is therefore calculated as:

$$P_f = 1 - e^{-RR \cdot L} \tag{5}$$

where RR=max(RR_{PGV}, RR_{PGD}), with RR_{PGV} and RR_{PGD} calculated as in Equation (4). Note that Equation (5) is a Poisson process and thus, is 'memoryless' disregarding any failures that may have occurred along the pipe in the past.

PROPOSED STRATEGY FOR PIPE VULNERABILITY ASSESSMENT

As already discussed, the study's goal is to propose a seismic vulnerability assessment methodology for water pipe networks, exploiting available data of everyday network failures due to sources other than seismic. Previous research has shown that survival analysis is a valuable tool for implementing methods for monitoring, repairing or replacing aging infrastructures and proactively devising strategies to keep the network in operation. Compared with failures caused by earthquakes, failures from non-seismic causes are more frequent and well distributed in time, whereas failures due to seismic effects occur intermittently and only when a major earthquake strikes. Thus, it is convenient to compile separately the data from the two failure causes. This approach is also close to the current practice, because usually it is the water agencies that maintain records of the everyday failure causes, whereas the seismic effects on the lifelines are usually given a more high-level attention by the civil protection agencies. Moreover, the approaches followed for seismic and non-seismic effects have distinct differences and therefore, it is not straightforward to post-process the data in a manner that allows to combine consistently pipe survival curves and vulnerability curves.

In our study, we combine the vulnerability curves suggested in the ALA guidelines with available survival curves that were compiled using network data available from theWater Board of Limassol (Cyprus). To this cause, we adopt a simplified engineering approach that allows us to quickly combine data that are not similar. Having in our disposal the pipe survival curves (e.g., Figure 1a) of $S(t)$ versus time (Equation (1)), we can calculate the survival probability of a pipe, depending on the NOPB and the pipe type (e.g., material, age and diameter). For this purpose we penalize the pipe

vulnerability function of Equation (4) by the ratio of the survival curve of the damaged case (NOPB≠0) over the undamaged pipe (NOPB=0). Therefore, after t days years, we define the ratio:

$$k(t) = S_{UD}(t) / S_D(t) \geq 1 \tag{6}$$

where subscripts 'UD' and 'D' stand for 'undamaged' and 'damaged', respectively. The modified pipe failure probability that now includes memory of past non-seismic failures is obtained after modifying Equation (5) as follows:

$$P_{\mathrm{f}}(t) = 1 - e^{-k(t) \cdot RR \cdot L} \tag{7}$$

Therefore, Equation (7) allows calculating the failure probability P_{f} of the pipe after t years given its NOPB metric, which is usually available from historical records. Figure 1b shows the pipe fragility as obtained from the first equation of Eq. (4), assuming K_1=1. Since $k(t)$ is the ratio of the damaged over undmaged survival curves, $k(t)$ becomes infinite when the denominator of Equation (6) becomes zero. The effect of $k(t)$ on the pipe fragility is shown in Figure 1b. As $k(t)$ increases, the failure probability takes larger values. For example, according to Figure 1b, a relatively small $k(t)$=5 value, for a moderate PGV of, say, 50cm/s will increase 50 times the RR. For small PGVs, the effect of $k(t)$ on the RR is less pronounced, whereas it practically remains constant as PGV increases.

RELIABILITY ASSESSMENT OF A WATER NETWORK

Once the failure probability, P_{f}, of every pipe is known, the performance of the network and its failure probability $P_{\mathrm{f,N}}$ can be assessed. Depending on the problem at hand, different approaches can be preferred. Perhaps the most significant parameter that affects the selection of the strategy to follow is how the network performance is measured and thus how the failure probability of the network is defined. In the simplest case, the network fails when it is not able to deliver water from its sources (inflow vertices) to every house connection (outflow vertices). Another, approach would consider the number of customers that are left without water. If such, rather simplified, network performance definitions are adopted, the performance of the network can be quickly evaluated using methods based on graph theory (Gibbons 1985). Alternatively, if the failure is defined with respect to hydraulic quantities, i.e., the hydraulic head in every house connection should not be less than a given minimum value, then hydraulic analysis of the network is required. Appropriate software is necessary in the latter case.

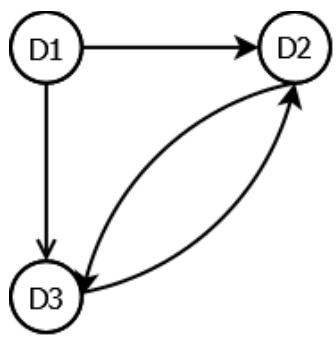


Figure 2 Example of a simple Graph.

We consider as failure of the network its inability to provide water to a consumer/house connection. Therefore, we define the failure probability as the probability of the network being unable to provide water from an inflow source vertex i to an outflow (e.g., house connection) vertex j. Inflow and outflow nodes are also called *sources* and *sinks*, respectively. If the failure probability to deliver water between i and j is $P_{\mathrm{f,ij}}$, the network reliability $R_{\mathrm{f,ij}}$ is defined as:

$$R_{\mathrm{f,ij}} = 1 - P_{\mathrm{f,ij}} \tag{8}$$

The Monte Carlo simulation (MCS) method is often employed when the analytical solution is not attainable and the failure domain cannot be expressed or approximated analytically. This is mainly the case in problems of complex nature with a large number of basic variables where other reliability methods are not applicable. If N_{H} is a large number, an unbiased estimator of the probability of failure is given by:

$$P_{\mathrm{f,ij}} = \frac{1}{N_{\infty}} \sum_{j=1}^{N_{\infty}} I\left(\mathbf{x}_j\right) \approx \frac{N_H}{N_{\mathrm{MCS}}} \tag{9}$$

where $I(\mathbf{x}_{\mathrm{j}})$ is a boolean vector indicating successful or unsuccessful simulations. For the calculation of $P_{\mathrm{f,ij}}$, a sufficient number of N_{MCS} independent random samples is produced using a specific probability density function for each component of the array $\mathbf{x}$. Therefore, N_{H} is the number of simulations where failure occurred, whereas N_{MCS} is the total number of simulations necessary to obtain an accurate estimation of the probability $P_{\mathrm{f,ij}}$. If a given accuracy δ_0 is required, the sample size can be approximately obtained using the formula:

$$N_{\mathrm{MCS}} = \frac{1}{P_{\mathrm{f,ij}} \delta_0^2} \tag{10}$$

Therefore, if the desired accuracy is δ_0=10% and the probability sought is of the order of 0.01, the required sample size N_{MCS} is $1/(0.1\times0.01^2)$=100,000 simulations. Equation (10) indicates that we must have a sufficient number of failed simulations, or in other words, the nominator N_{H} in Equation (10) must be sufficiently large in order to have a reliable estimation of $P_{\mathrm{f,ij}}$. For our problems, the reliability estimations were not found sensitive to the network size (i.e. number of pipes, house connections, etc). However, when the dimension of the problem is large, depending on its complexity, the necessary number of simulations may vary and thus a more elaborate sampling scheme may be necessary. In all, significant computational effort may be required, depending on the order of the probability sought and the properties of the problem at hand.

Given: inflow node *i*, outflow node *j*
for all pipes **do**
- determine if the pipe fails (use the pipe P_{f} & assume binomial distribution).
- **if** the pipe fails **THEN** remove it from the graph

end for
set $countFailedSimulations = 0$,
for N_{MCS} **do**
- determine if *i* and *j* are connected (use Dijkstra's algorithm, or Eq. (2))
- **if** there is no connection **THEN**
- set $countFailedSimulations = countFailedSimulations + 1$

end for
$\widehat{P}_{\mathrm{f,ij}} = countFailedSimulations \big/ N_{\mathrm{MCS}}$

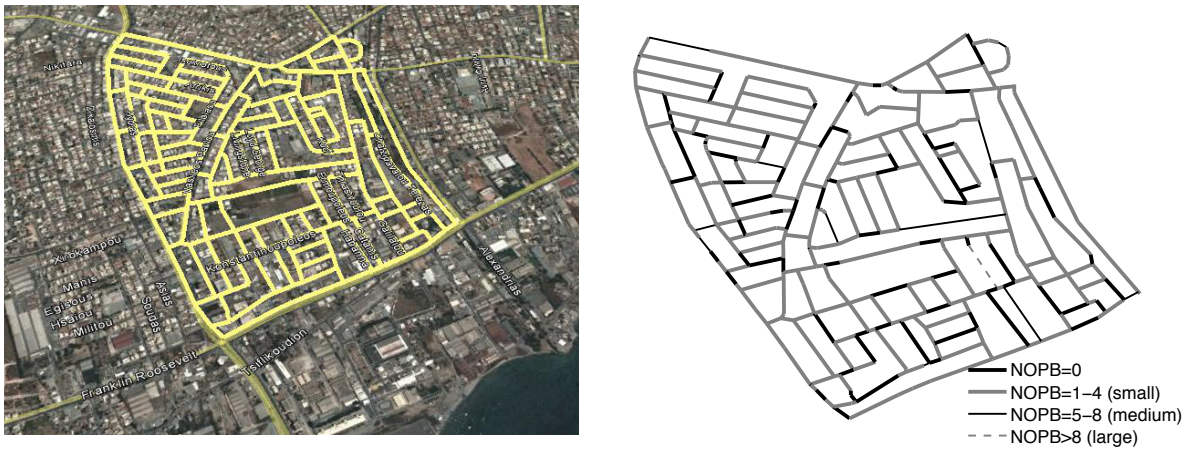
Figure 3 Flowchart of MCS

When MCS is adopted on pipe networks, the calculation is based on reducing the network topology, that is, removing pipe segments, which are assumed as failed. The flowchart of the Monte Carlo method implemented here is shown in Figure 3. For every simulation, a state vector is produced. In this vector, two states can be considered for every pipe: *0-state*, which refers to a failed state with probability $P_{\mathrm{f,ij}}(t)$ (Equation (7)) and *1-state* that corresponds to non-failure with probability 1-$P_{\mathrm{f,ij}}(t)$. Once a state vector is obtained, the failed pipes are removed from the network. Using common graph algorithms, we can determine whether a path between vertices *i* and *j* exists, thus allowing water flow

delivery from node i to node j. In all our applications, a standard Dijkstra algorithm (Gibbons 1985) gave quick and robust calculations. If at least one path exists the simulation is successful, otherwise it has failed. The network reliability R_{ij} can then be evaluated by dividing the number of successes with the total number of simulations performed (Eq. 10).

NUMERICAL EXAMPLE

The second case study considered, is a district metered area (DMA) of the water network of the city of Limassol, Cyprus (Figure 4). The city network is clustered into DMAs, which are areas with one inflow vertex. This practice allows the Water Board to isolate damage in the network within finite domains (DMAs) and then handle any problem that may occur within a DMA without affecting the rest of the city network. Figure 4a shows the aerial view of the city together with the graph model used for simulating the network, which has been produced using available GIS data. GIS is a powerful tool for creating the graph network and obtaining details regarding the properties of the network, e.g. number of consumers in every house connection. However, its use should be careful and the idealisation made has to be as close as possible to the actual geometry of the network. Errors in this idealisation may considerably affect the outcome of the analysis and introduce bias. For example, if the pipe length is not correctly modelled, the pipe failure probabilities will vary considerably when calculated using Eqs. (5) and (7).



(a) (b)

Figure 4 (a) aerial view of the DMA studied, (b) Graph model of a DMA of the water network of Limassol, Cyprus. The plot shows also the NOPB of every pipe.

In total, the water network consists of 337 pipes/edges and 259 vertices/nodes and covers an area of 780×450m. The total pipe length is 23,724 m, and according to the records of the Water Board of Limassol (Cyprus), the number of consumers served by the DMA studied is 6,585 people. On average every node serves approximately 25-30 consumers, while the maximum number of consumers per node is 120. The pipe material is asbestos cement (AC) and is the same for every pipe. Since the elevation is practically constant throughout the network, we assume that the network is bi-directional. Figure 4b shows the topology of the network and the number of previous breaks of every network pipe/edge. The pipe survival curves were those of Figure 1a, which were based on actual data obtained from the Water Board of Limassol, Cyprus for this DMA.

For buried pipelines, seismic hazards can be classified as either wave propagation hazards or permanent ground deformation (PGD) hazards, e.g. 1985 Michoacan earthquake in Mexico City. Typically pipeline damage is due to a combination of hazards. According to O'Rourke *et al.* (1985), roughly half of the pipe breaks in the 1906 San Francisco earthquake occurred within liquefaction-induced lateral spreading zones while the other half occurred over a somewhat larger area where wave propagation was apparently the dominant hazard. Thus, PGD damage typically occurs with high damage rates in isolated areas of ground failure, while wave propagation damage occurs over much larger areas, but with lower damage rates (O'Rourke 2003). This is also evident from the repair rates

of Eq. (4), where adopting typical values of PGV and PGD, the PGD equation will give rates of a different order of magnitude.

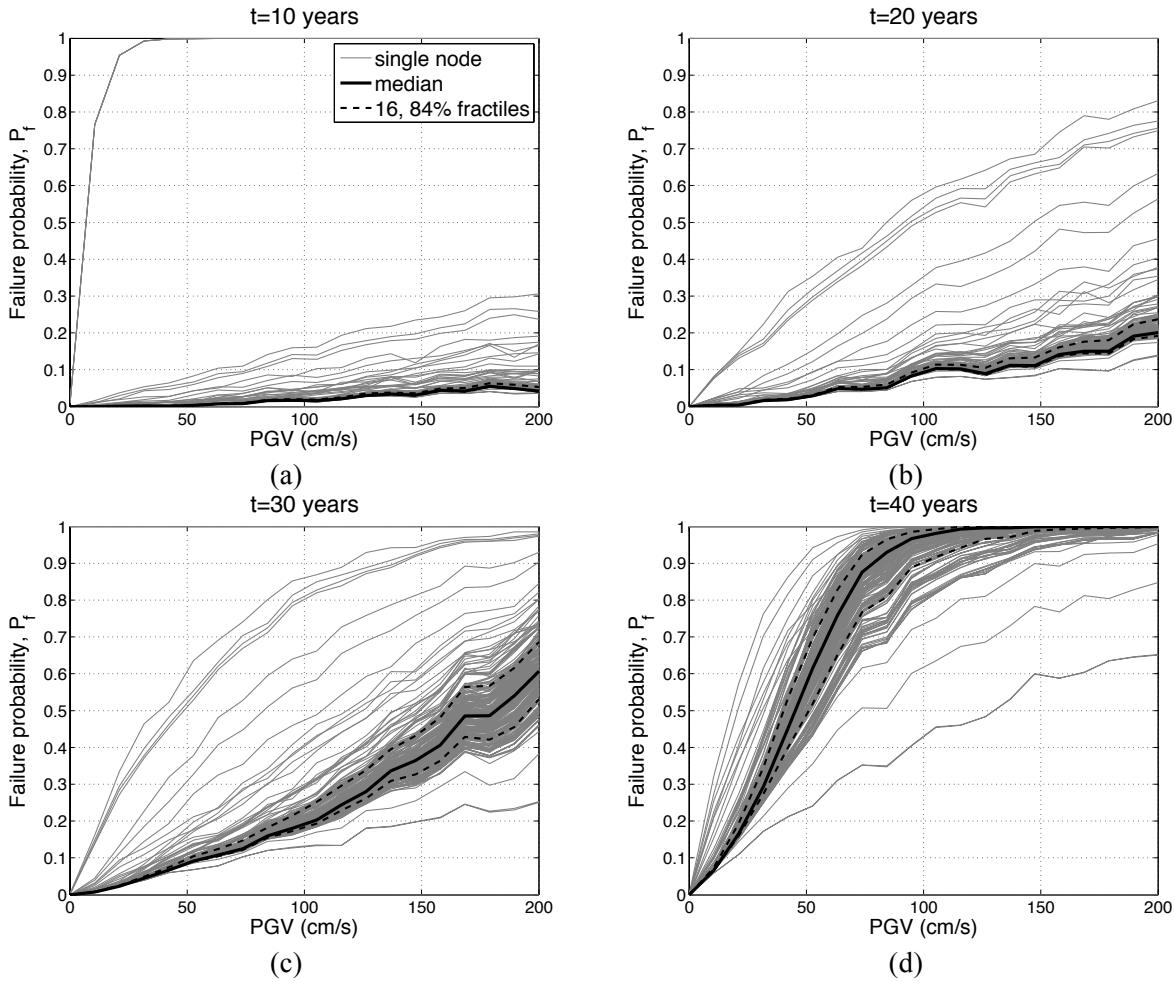


Figure 5: (a) Fragility curves of every house connection versus peak ground velocity, after: (a) 10 years, (b) 20 years, (c) 30 years and (d) 40 years.

Based on the above observations, we consider two seismic scenaria. In the first scenario, damage is only due to wave propagation. Being consistent with the seismic hazard in the island of Cyprus which is mainly controlled by distant and moderate magnitude events, it is valid to assume uniform seismic intensity throughout the DMA. Here we measure seismic intensity with the aid of peak ground velocity (PGV). In the second scenario both PGV and PGD occur, but PGD is isolated in a small part of the network. For both scenaria, we produce fragility curves for every outflow node *j*. The outflow node is always the same. If more that one inflow vertices exist, the network failure probabilites can be easily obtained by repeating the proposed prosedure for every inflow vertex and appropriately combing the final node failure probabilities.

Figure 5 shows the fragility curve of every vertex with respect to peak ground velocity (PGV) for four time instances measured from the installation of the network. As PGV increases we calculate the pipe failure probability using Eq.(7) and the corresponding node probability using Eq.(9). Therefore, the gray lines correspond to the probability of water being able to reach the corresponding valve, while the black lines are the median (50% percentile) and the 16% and 84% percentile curves, which are shown to provide a measure of the overall condition of the network.

Figure 5a,b show the fragility curves after 10 and 20 years of network operation, respectively. Since the network would be of "young age", the mean fragilities lie below 0.2, even for considerably high PGV values, e.g. PGV≥200cm/s (Figure 5a). Still, some house connections are vulnerable and their failure probability may exceed 40%. Moreover, there are nodes whose failure probability is very high. This is due to the fact that these nodes are connected with the inflow source through pipes that

are connected in series, thus if any of the connecting pipes fails the water will not be able to reach them. In this case, the remedy will be to create conditions of redundancy by forming alternative water paths. Figure 5b,c,d have the well-known form of fragility curves, showing that the network vulnerability increased as the time passes and PGV increases. Note that since the construction of the pipes is made at a present time, the NOPB values are kept constant. Actually, NOPB will also vary as time passes, probably increasing the network vulnerability, but the prediction of survival analysis is based on the data available at the present time and therefore this effect is not considered in our analyses. Again in Figure 5b,c there are stray lines away from the average, indicating that the vulnerability of some house connections may considerably differ from the average and thus the interpretation of the analysis should also be done on a node by node basis and not rely purely on global metrics on the DMA level. After forty years of operation (Figure 5d), even a relatively moderate PGV (≤50cm/s) will lead to high failure probabilities and therefore extensive damage on the network.

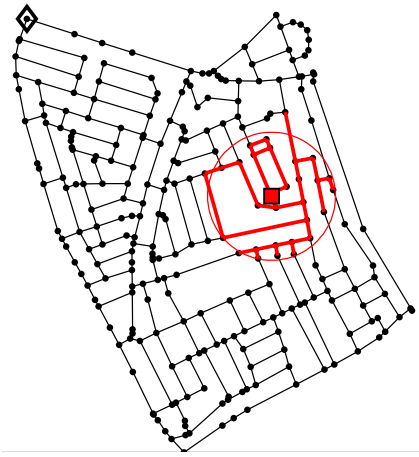
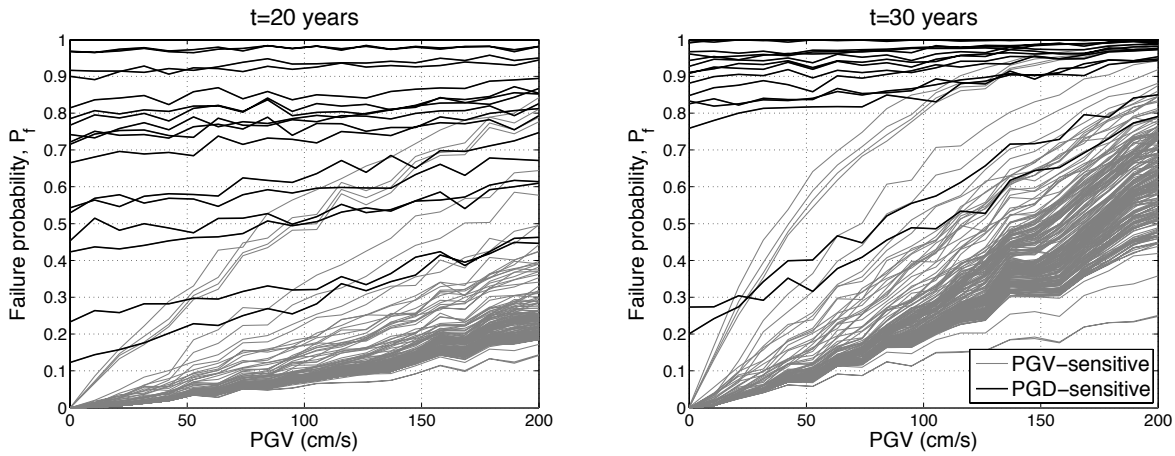


Figure 6 (a) Water distribution network and the area where a permanent ground deformation equal to 12.6cm (5 in) is imposed. The circel shows the affected area and the square is its center.



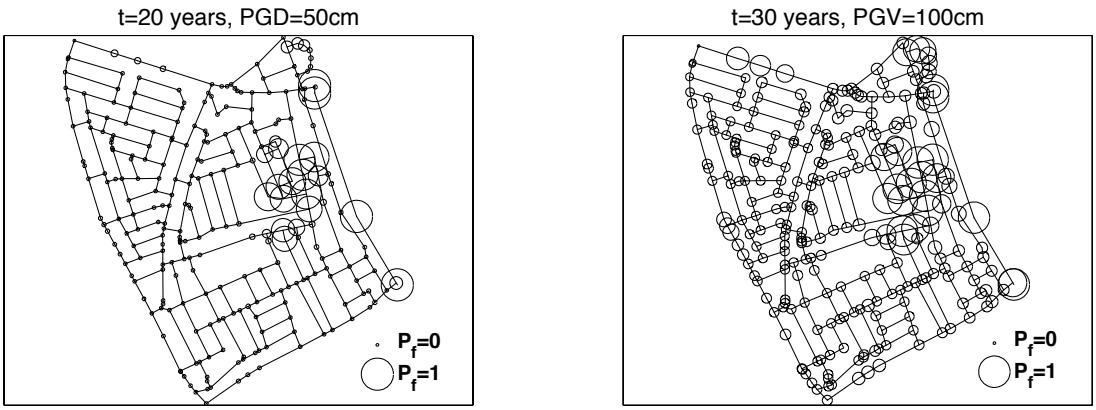
(a) (b)

Figure 7 (a) Fragility curves of every house connection versus peak ground velocity when a permanent ground deformation is imposed, afer: (a) 20 years, (b) 30 years.

In the second scenario considered, we assume a permanent ground deformation (PGD) due to a random cause. The PGD affects a wide area of radius equal to 200m around a point shown in Figure 6 with a red square. The deformation is assumed to be constant and equal to 12.6cm (5in). In Figure 6 we also show with a thick red line the pipes that are affected by the imposed PGD. The fragility curves of every vertex for t=20 years and t=30 years are shown in . It is evident from the plot, that there are nodes whose vulnerability is considerably higher that the rest. More specifically, looking and PGV=0 cm/s there are nodes whose probability is larger than zero. The vulnerability of these nodes is

governed by PGD and for visual purposes we show them with solid black lines and in the legend of are denoted as "PGD-sensitive".

Figure 8 shows the geographical distribution of risk after *t*=20 and 30 years, for PGV values equal to 50 and 100 cm/s, respectively. It is shown that "PGD-sensitive" are the pipes in the vicinity of the PGD, while the risk in the rest of the network is not affected considerably. The grey lines () correspond to vertices whose vulnerability is "PGV-sensitive". For the *t*=30 years case, the "PGD-sensitive" curves start from high probability values, and quickly approach 1, while for *t*=20 years a larger dispersion is observed. In any case, the "PGD-sensitive" curves are also affected by the increase of PGV (although with a smaller rate), since they operate within a network that combines PGV and PGD-sensitive components. Moreover, when considering both PGV and PGD the practice of producing average curves (e.g. Figure 5) is not useful, since the probabilities vary considerably and depend on the location of the node with respect to where the permanent ground deformation occurred.



(a) (b)

Figure 8 (a) Geographical distribution of failure probabilities: (a) *t*=20 years and PGV=50cm/s, (b) *t*=30 years and PGV=100cm/s.

ACKNOWLEDGEMENTS

This research has been co-financed by the European Union (European Social Fund – ESF) and Greek national funds through the Operational Program "Education and Lifelong Learning" of the National Strategic Reference Framework (NSRF)-Research Funding Program: THALES. Investing in knowledge society through the European Social Fund.

CONCLUSIONS

A general-purpose methodology for the reliability assessment of water pipe distribution networks has been presented. The proposed methodology builds on the framework of the ALA guidelines and includes the probability of failure due to non-seismic causes, as measured during the everyday operation of the network. The more frequent non-seismic failures are typically repaired immediately after the damage is reported to the water agency and result to increasing the future vulnerability of the damaged pipe. This sort of information is often available by water agencies and can be post-processed to provide the pipe survival curves. Using survival analysis, we propose a novel methodology for the vulnerability assessment of ageing water networks. Once the pipe failure probabilities are known, the reliability of the water network can be calculated using numerical (Monte Carlo) simulation methods.

REFERENCES

ALA (2001). "American Lifelines Alliance: Seismic Fragility Formulations for Water Systems-Guideline and Appendices".

Ballantyne D (1990) "Earthquake loss estimation modeling of the Seatle water system", *Technical Report 886005, supported by U.S. Geological Survey Project Number 14-08-0001-G1526*, Kennedy/Jenks Consultants.

Christodoulou SE (2011) "Water Network Assessment and Reliability Analysis by Use of Survival Analysis," *Water Resources Management*, 25: 1229-1238.

Christodoulou SE, Deligianni A (2010) "A neurofuzzy decision framework for the management of water distribution networks," *Water Resources Management,* 24(1):139–156.

Christodoulou SE, Ellinas G. (2010) "Pipe routing through ant colony optimization," *ASCE Journal of Infrastructure Engineering,* 16(2): 149–159.

Gibbons A. (1985) Algorithmic Graph Theory, Cambridge University Press. 13.

Henley EJ, Williams RA (1973) Graph Theory in Modern Engineering, Academic Press, New York, USA.

Hintze J (2001). NCSS and PASS. Number Cruncher Statistical Systems. Kaysville, Utah.

Hosmer D, Lemeshow S, May S. (2008) Applied Survival Analysis: Regression Modeling of Time To Event Data New York, NY, US|USA: A Wiley-Interscience Publication, John Wiley & Sons Inc.

Kaplan EL, Meier P (1958). "Nonparametric estimation from incomplete observations," *J. Am. Stat. Assoc.*, 53; 457481. 18.

O'Rourke TD, Grigoriu MD, Khatter MM (1985). "A State of the Art Review: Seismic Response of Buried Pipelines", in Decade of Progress in Pressure Vessels Technology, Ed. C. Sundararajan, *American Society of Mechanical Engineers*, New York.

O'Rourke M.J. (2003) "Buried Pipelines in Earthquake Engineering Handbook", in New Directions in Civil Engineering (Eds).

Renschler C, Frazier A, Arendt L, Cimellaro GP, Reinhorn AM, Bruneau, M (2010). "Framework for Defining and Measuring Resilience at the Community Scale: The PEOPLES Resilience Framework. *MCEER Technical Report MCEER-10-006, pp. 91, University at Buffalo (SUNY)*, The State University of New York, Buffalo, New York.

Romero N, O'Rourke TD, Nozick LK, Davis CA (2010). "Seismic hazards and water supply performance", *Journal of Earthquake Engineering*, 14(7): 1022-1043.