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WATER DISTRIBUTION NETWORKS PERFORMANCE UNDER SEISMIC RISK AND AGEING DETERIORATION

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MEMORIA PARA OPTAR AL TITULO DE INGENIERO CIVIL

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RESUMEN DE LA MEMORIA PARA OPTAR
AL TÍTULO DE MAGÍSTER EN CIENCIAS
DE LA INGENIERÍA
POR: **NICOLÁS CÁRDENAS BASCUÑÁN**
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La presente tesis tiene como objetivo el generar una metodología de análisis de riesgo sísmico para una red de agua potable deteriorada por el tiempo. Esta metodolgía se divide en 6 partes: i) análisis hidráulico de la red de agua, ii) muestreo de eventos sísmicos, iii) estimación de la tasa de fallas, iv) estimación del deterioro temporal, v) evaluación del performance de la red y vi) evaluación de riesgo de la red.

Se trabaja con 9 métricas de performance de 3 tipos. Las del tipo hidráulico corresponden a las métricas de: i) confiabilidad basada en presiones, ii) confiabilidad basada en demanda satisfecha, iii) vulnerabilidad, vi) resiliencia y v) sostenibilidad. Las métricas topológicas o de conectividad son: i) coeficiente de mallado y ii) betweenness centrality. Mientras que las de entropía son: i) grado entrópico y ii) grado entrópico modificado.

El desempeño de la red puede representarse con un modelo probabilístico, ya que la ubicación de las tuberías dañadas no es conocida. Esto convierte al problema en un análisis complejo, dependiente del evento sísmico, estado de deterioro y distribución de las fallas dado un evento sísmico. Por ello se desarrolla una metodología que descompone este problema complejo en tres fases. La primera, consiste en determinar la distribución del desempeño de la red para todas las métricas, conocido una tasa de falla (es decir, cantidad de fallas del sistema). La segunda, es generar el muestreo de eventos sísmicos y la velocidad máxima de suelo (PGV) que actúa en el sistema. Finalmente, mediante correlaciones entre la velocidad máxima de suelo y la tasa de falla se puede relacionar cada evento sísmico a una tasa de falla en el sistema y a su correspondiente distribución de desempeño. Para el caso específico de este trabajo se utiliza el desempeño medio y el 90 % más desfavorable.

Para probar la metodología propuesta se utiliza una red de agua potable topológicamente real, que consiste en 2 fuentes, 2156 nodos, 2422 tuberías con una extensión aproximada de 100 km. Para efectos sísmicos se sitúa esta red en la ciudad de Viña del mar Chile. Para las métricas hidráulicas, dada la existencia de un evento sísmico superior a una magnitud 7, se obtiene el desempeño medio y 90 % más desfavorable a lo largo del tiempo.

Los resultados demuestran una gran influencia del envejecimiento en el desempeño. Se obtienen también las rutas criticas del sistema mediante las métricas de grado entropico, grado entropico modificado y betweenness centrality. Finalmente, se concluye que la metodología es aplicable a una red topológica real y entrega herramientas de proyección del estado post evento sísmico e identificación de lementos elementos mas críticos. Esta metodología puede ser empleada para generar planes de mitigación y/o recuperación ante eventos sísmicos.

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The objective of this thesis is to generate a seismic risk analysis methodology for a drinking water network deteriorated by time. This methodology is divided into 6 parts: i) hydraulic analysis of the water network, ii) seismic event sampling, iii) failure rate estimation, iv) age deterioration estimation, v) network performance evaluation and vi) network risk assessment.

We work with 9 performance metrics of 3 types. The hydraulic metrics correspond to: i) reliability based on pressures, ii) reliability based on satisfied demand, iii) vulnerability, vi) resilience and v) sustainability. The topological or connectivity metrics are: i) meshing and ii) betweenness centrality. The entropy metrics are: i) entropic degree and ii) modified entropic degree.

A water network performance can be presented with a probabilistic model since the location of damage pipes is not known. This makes the problem a complex analysis, dependent on the seismic event, state of deterioration and distribution of failures given a seismic event. We developed a methodology that decomposes this complex problem into 3 phases. The first is to determine the distribution of network performance for all metrics, given a known failure rate (i.e., number of system failures). The second is to generate a sampling of seismic events and the peak ground velocity (PGV) that acts on the system. Finally, through correlations between the peak ground velocity and failure rate, each seismic event can be related to a failure rate and its corresponding performance distribution. For the specific case of this work, the average performance and the 90 % worst case are used.

To test this implementation a topologically realistic drinking water network is used, consisting of 2 reservoirs, 2156 nodes, 2422 pipes with an approximate extension of 100 km. To define a seismic environment, this network is located in the city of Viña del Mar, Chile. For the hydraulic metrics, given the existence of a seismic event greater than moment magnitude 7, the average and 90th percentile performance over time are obtained. The results show a strong influence of aging on performance. The critical paths of the system are also obtained by using the entropic degree, modified entropic degree and betweenness centrality metrics.

Finally, it is concluded that the methodology is applicable to a real topological network and provides tools for projecting the post-seismic state and identifying critical elements. Furthermore, this methodology can be used to generate mitigation and/or recovery plans for seismic events.

"Sometimes life doesn't let you choose your battles.
Just the company you keep."

Fredrik Backman

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Chapter 1

Introduction

1.1. Background

Water distribution networks (WDN) are one of the most critical infrastructure in a city. Their proper operation not only guarantees supply but is indirectly a key factor in the development of a country by playing a fundamental role in terms of sanitation and social equity [1]. Their resilience, the ability to withstand and recover after a disaster such as an earthquake, is essential given the interdependence with other lifelines (telecommunication, power supply, gas) and their importance for emergency institutions, as firefighters and hospitals [2][3][4].

Natural hazards such as floods, hurricanes, fires or earthquakes and their potential consequences over the water distribution network vary depending on the location of the event, its intensity and the elements' vulnerability [5]. Given their enormous destructive potential, the extension of the areas affected, and the impossibility of forecasting their occurrence, earthquakes are one of the most serious hazards [6]. The devastation caused by earthquakes to water distribution networks and its aftermath have been wildly documented for cases such as the 1995 Nanbu, the 1994 Northridge or the 2010-2011 Christchurch earthquakes [7][8][9], where failure of the transmission system, breakage in distribution and service lines were the main causes of service disruption. Moreover, water distribution networks around the world are deteriorating, causing lower quality, breakage and leakage. Some of the pipelines in major cities have had a lifespan of more than 100 years, and although replacement is in process in most cities, daily operation is still done with deteriorated pipelines.

Reliability assessment studies of drinking water networks in the face of hazard events have introduced the time variable through a pipeline survival function as a way to capture the aging [10][11]. This function modifies the probability of survival of the pipelines in front of seismic loads according to their age. On the other hand, the survival functions used in these studies do not modify internal variables of the pipeline, and therefore do not impact the water distribution. Furthermore, the survival functions are not developed based on pipelines damaged by seismic events, but rather are built based on the time between failures [12] [13].

The internal deterioration of the pipes and the increase in energy losses have been attributed to the increase in the roughness coefficient over time. This deterioration has been defined in several studies with functions of increase in the roughness coefficient in the pipe [14][15][16].

The increase in energy losses has an inverse relationship with the magnitude of the network pressures. Which, in turn, are directly related to the ability to overcome adversities such as pipe ruptures [17].

The effect of deterioration and seismic vulnerability on the performance of water distribution networks must be assessed to define courses of action regarding prevention and mitigation policies. This thesis aims to conduct a seismic risk evaluation to assess system reliability under seismic loading and ageing. The authors propose a seismic risk framework for lifelines, applied to water distribution systems. Different networks are used to assess resilience, performance and critical elements.

1.2. Motivation

There are several examples of water networks affected as a result of a seismic event in areas where the event seemed foreseeable. On February 27, after an 8.8 Mw earthquake, the Gran Concepción area lost 90 % of its drinking water distribution capacity and did not recover it 85 % capacity until 3 weeks later, leaving substantial monetary losses and affecting the lives of thousands of people [18].

Understanding the behavior of drinking water distribution networks, when these are subject to seismic events, is the key to knowing the most vulnerable zones and generating an action plan for the creation of a more resilient network. One of the ways to estimate the damage of the network after a seismic event, corresponds to the capacity to predict the amount of failures and their location. Currently, there are studies that define the failure rate per kilometer as a function of the characteristics of the terrain (topography, probability of liquefaction), pipes (diameter and material) and the peak ground velocity [19].

In addition to the deterioration of the system as a result of a seismic event, there is a constant deterioration over time associated with pipes aging. The best way to illustrate aging is through increased pipe roughness and higher energy loss of water flow. Different studies modeling the increase in roughness are presented in the literature review section in order that they can be adopted according to the context of each study or research.

To determine the network performance, various metrics have been developed over time. These metrics are generally studied in synthetic networks. Therefore, it is important that they are tested on topologically real networks to know their ability to capture the real performance of the network.

The present work aims to understand how increased energy losses influence the reliability, vulnerability and resilience of a drinking water network during seismic events. In addition, it aims to provide tools that allow the generation of a better action plan, by presenting critical routes obtained through topological and entropic metrics.

Last but not least, the aim is to provide tools for future work in the area of risk management. For this reason, several studies of roughness increase and the relationship between repair rate and PGV are presented. The choice of the models (roughness increase and relationship between repair rate and PGV) should be chosen through knowledge of the seismotectonic scenario, soil type and pipe properties.

1.3. Objectives

1.3.1. Main objectives

Generate a methodology of seismic risk analysis for drinking water networks deteriorated by time.

1.3.2. General Objectives

- Propose a metric for network performance that reflects the process of distribution and deterioration.
- To implement a methodology that allows to make hydraulic balance based on demands that considers the existence of negative pressures and damages in the nodes.
- Define a methodology to generate seismic scenarios and perform sensitivity analyses to reduce computational time.
- Find a metric that provides comprehensive insight into the performance.

1.4. Previous research

Being able to define the performance of a water network after a disruptive event allows a better understanding in terms of better management of resources for risk mitigation or recovery of the area after the event. This is why several researches have been conducted with the purpose of defining the performance as a function of variables such as pressure, satisfied demand or energy surplus in the system. The following section presents a brief review of these works, with the purpose of introducing background information prior to the development of this thesis.

Since then, research has taken two main approaches. The first one consists in the improvement and definition of the most critical pipelines. The second one corresponds to the prediction of performance after the risk event.

1.4.1. Renewal cycle and most benefit renewal methodology

In the pursuit of building a more reliable network and using the metrics of system serviceability index (SSI), damage consequence index (DCI) and improvement benefit index (UBI), Wang et al. develop a methodology that decreases the probability of failure of a pipe or a set of pipes in order to know the network performance in improved reliability. They develop a methodology that decreases the probability of failure of a pipe or a set of pipes in order to know the performance of the network at improved scenario. In this way, it is defined which pipes whose replacement or upgrading has a greater benefit for the network performance [20].

On the other hand, Mancuso et al. develop a methodology to identify an optimal set of inspections of network elements whose renovation can reduce risk and cost, and also optimize inspection of degraded elements and maintenance. The methodology used a Multi-Attribute Value Theory, systematic methodology for evaluating decision alternatives with regard to multiple objects, and results are given in terms of robust portfolio management core index (pipe core index vs sorted pipes), portion of renovated pipes vs pipe states, and correspondence between failure likelihood and degradation state probability [21].

1.4.2. Performance forecasting methodology

In 1978 the first probabilistic approach was developed by Shinozuka et al. and included several causes of damage: ground motion associated to wave propagation, fault movement, liquefaction, landslide, and interference with other under-and-above-ground structures. Shinozuka et al. develop a procedure for estimating system serviceability, this methodology is defined in Figure 1.1. As it includes probability of seismic hazard, it follows a probability approach, being the first analysis conducted in this fashion, but it does not account for the hydraulic analysis of the damaged system [22].

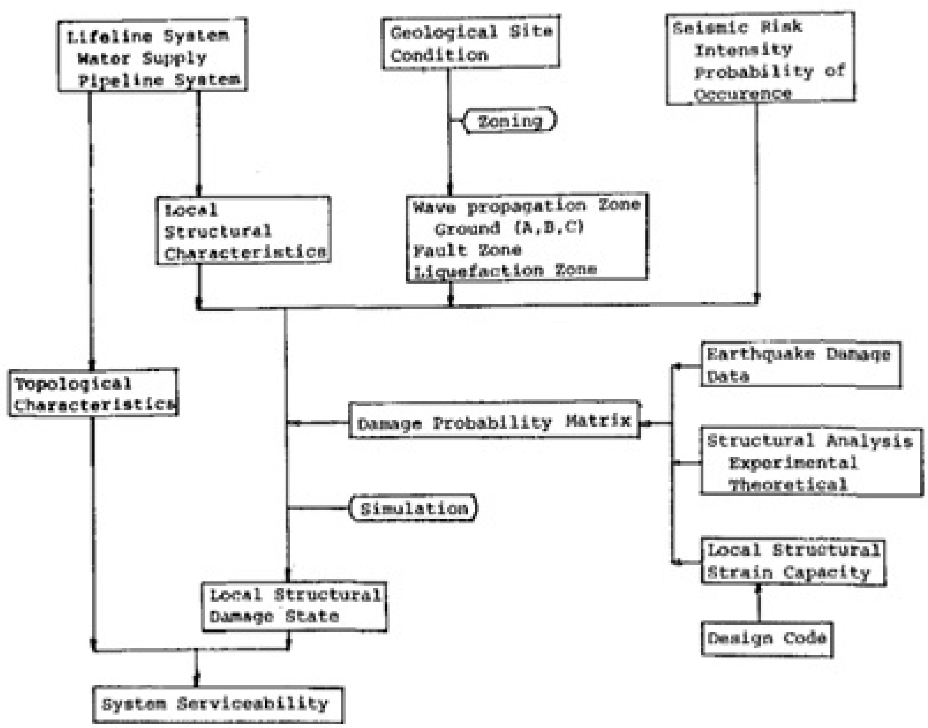

Figure 1.1: System serviceability diagram

In a more complex work, Ballantyne and Waisman applied three risk analysis: i) hazard/risk screening and risk quantification for individual components for multiple hazards, ii) fault tree analysis for supply system for high-risk hazards, iii) hydraulic analysis of damaged distribution system. Outage was based on repair time. They did a fault tree analysis bases on the probability of supplying water (considering the complement probability of not being functional), a connectivity analysis was conducted including the earthquake effect, to show flow versus the probability of supplying at least a certain amount [23].

In a similar way, AWWA devolps a framework for Risk analysis and Management for Critical Asset Protection (RAMCAP). Their risk analysis consists of: i) asset characterization, ii) threat characterization, iii) consequence analysis, iv) vulnerability analysis, v) threat analysis, vi) risk/resilience analysis, vii) risk/resilience management. Calculation of risk for each threat is computed as Risk=Consequences$\times$Vulnerability$\times$Threat Likelihood, where consequences are expressed in terms of the number of fatalities, number of serious injuries, financial losses to the owner, and economic losses to the metropolitan region in which the facility operates; vulnerability is the likelihood, given that a threat happens, that once the asset has been impacted by the threat results in the consequences. The current level of resilience is estimated considering connectivity, interdependencies, complexities, preparedness, continuity of operations, and recovery [24].

In order to incorporate the aging effect Yoon et al. incorporate a survival function, which is time dependent and modifies the pipe's failure probability. The framework for seismic risk assessment of urban water transmission networks consist in Graph theory to find a performance indicator such as connectivity and flow reliability. They used connectivity loss and serviceability ratio, they used a spatially correlated seismic attenuation law to estimate PGA and then estimated the repair rate and the failure probability proposed by ALA (2001). Then, this repair rate and failure probability were modified to be expressed as the ratio of survival functions of the intact pipes over the one of the damaged, times the repair rate. Moreover, they included fragility curves for water treatment plants, pumping plant, electric power substation, and storage tanks. Regarding deterioration, they included deterioration of each pipeline using a survival function and then fragility curves for 20 and 30 years. The results were presented as 3D plot in terms of magnitude, time and connectivity loss or serviceability ratio [11].

Chapter 2

Probabilistic Framework for Risk Quantification

The risk quantification of water pipe networks start with the adoption of three essential models: (1) a network state predictor, (2) a failure pipe model and (3) a performance model based on the network state [25]. The first two are combined to predict the state of the network when some elements of the network present failures, while the third model takes this network state to evaluate the performance by employing different metrics that are used an indicator of the network serviceability. Note that the performance metrics are usually adopted based on the particular interest of the stakeholders. In the illustrative example presented later in Section 5, different metrics are presented and discussed for a real water network. Within this framework, the different uncertainties that arise in each individual model should be considered and propagated. The general scheme of the framework is presented in fig.2.1 and it will be fully described in the following subsections.

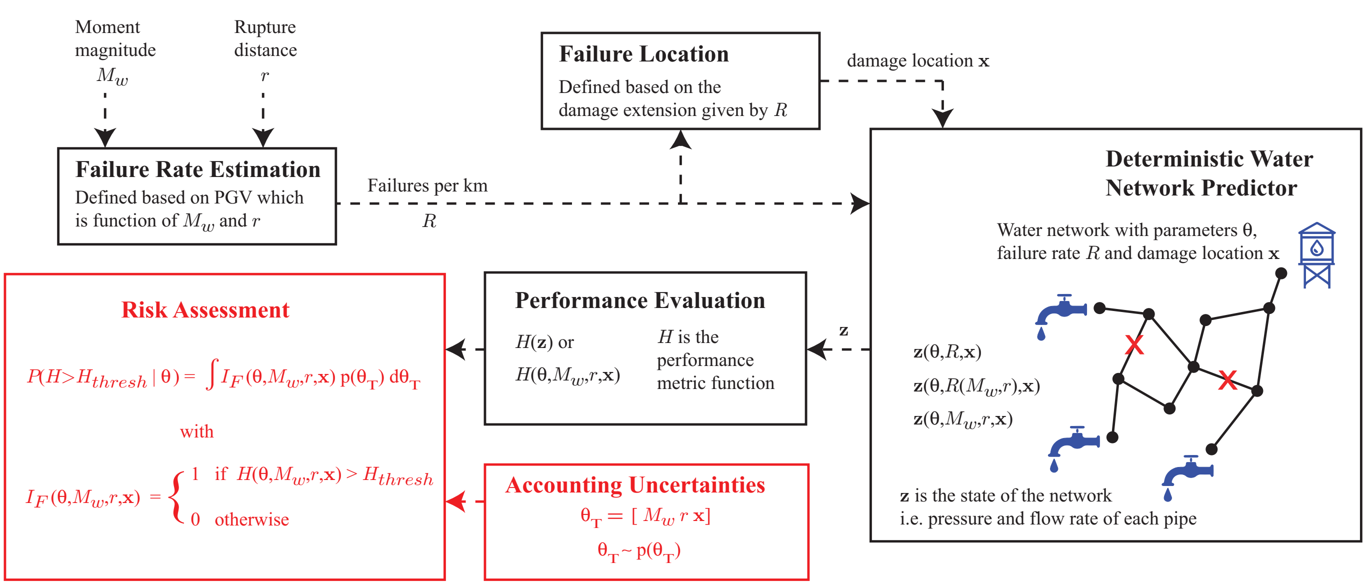

Figure 2.1: Proposed risk quantification scheme for water pipe networks.

2.1. Modeling Water Network Performance

The performance of the water distribution network is obtained from the analysis of the output variables of the hydraulic analysis. Depending on the metric on which the performance is based, variables such as pressure head of the nodes, flow in pipes, flow delivered at the nodes may be required. The input variables of the hydraulic analysis correspond to the network topology (reservoirs, nodes and pipes connecting them), demand flow at the nodes, valves, pumps, pipe diameter, pipe roughness, etc. While the output variables are the flow in the pipes, pressure head at the nodes and flow delivered by the nodes. The demand driven hydraulic analysis is modeled as the minimization of energy loss subject to required flow delivery compliance at the nodes. On the other hand, the pressure driven hydraulic analysis adds the constraint that the pressure at all nodes must be greater than or equal to 0.

2.1.1. Construction and resolution of the closed network problem

A closed network is one in which the ducts that make it up are closed to form a circuit. This is the case of drinking water networks in cities.

The solution to the problem is based on two types of equations: the node flow continuity equation and the energy loss equation. The continuity equation in the node indicates that the sum of the incoming and outgoing flows is zero and is presented in Eq. 2.1. The energy loss equation indicates that the sum of loss energy between two nodes considering all the possible path is zero and it is presented in Eq. 2.2. The loss between two nodes is defined in Eq. 4.1 while the constant of loss per unit of square flow is defined in the equation 2.4.

$$\sum_{j \in i} Q_{ij} + Q_i = 0 \tag{2.1}$$

$$\sum_{1}^{k} h_{ij} = 0 \tag{2.2}$$

$$h_{ij} = a_{ij} Q_{ij}^2 \tag{2.3}$$

$$a_{ij} = \frac{8 f_{ij} L_{ij}}{\pi^2 g D_{ij}^5} \tag{2.4}$$

Where Q_{ij} is the pipe flow rate between nodes i^{th} and j^{th}, Q_i is the pipe flow rate in the node i^{th}, f_{ij} is the pipe frictional constant between nodes i^{th} and j^{th}, L_{ij} is the pipe length between nodes i^{th} and j^{th}, D_{ij} is the pipe diameter between nodes i^{th} and j^{th} and g is the gravity constant. As a convention, outflow is considered positive.

Since h_{ij} is the loss corresponding to the section between the nodes i and j. This can also be written as a function of the difference in pressure head: 2.5.

$$h_{ij} = H_j - H_i \tag{2.5}$$

Where H_i is the pressure head in the node i^{th}.

Then, replacing the equation 2.5 in 2.3 it is possible to write the pipe flow rate as follows:

$$Q_{ji} = \left(\frac{H_j - H_i}{a_{ij}} \right)^{\frac{1}{2}} = C_{ij}(H_j - H_i)^{\frac{1}{2}} \tag{2.6}$$

Where $C_{ij} = \frac{1}{a_{ij}}^{\frac{1}{2}}$ is a constant.

Considering the flow in the node as known, we have n equations 2.1. Where n corresponds to the total number of nodes. Arranging these equations as a matrix, the following equation is written:

$$[K]\{H\} = \{Q\} \tag{2.7}$$

Where H is the head pressure vector for all nodes, Q is the flow vector for all nodes, k_{ii} is the diagonal term of $[K]$ and correspond to the sum of C_{ij} if $i \in j$ and k_{ij} is the non-diagonal term of $[K]$ and correspond to $-C_{ij} if i \in j$ and 0 otherwise.

$[K]$ is symmetrical and definite-positive. So it is possible to obtain the pressure heights in all the nodes through the inversion of $[K]$.

$$\{H\} = \{Q\}[K]^{-1} \tag{2.8}$$

The resolution of this system of equations is obtained through the definition of an initial pressure head vector H. With this vector the flows, losses and again the pressure heads are calculated.

2.2. Risk Quantification

Seismic-induced damage to buried structures, unlike superstructures above ground, is dominated by permanent ground deformation (PGD) and peak ground velocity (PGV). A fundamental requirement for evaluating the seismic performance of a water service is the ability to quantify the damage potential of the components as a function of the level of seismic hazard [26].Therefore, PGV and PGD play a fundamental role, since they represent the behavior of the soil as a function of the characteristics of the seismic event (such as moment magnitude (M_w) and rupture radius (r)) and local characteristics where the drinking water network is located. Repair rate (R) is widely used by various authors to relate the PGV/PGD with the number of failures per kilometer. This parameter has been collected for several seismic events, such as 2010 Darfield, 2011 Christchurch [27], 1995 Hyogoken-nanbu [19], 1994 Northridge and 1989 Loma Prieta [26], and different regressions have been computed to relate R as a function of PGV/PGD, ultimately function M_w and r.

The integration of damage in the network is done through the creation of nodes in the middle of the pipes to represent leaks. The nodes that represent leaks do not have a base demand and are assigned an emitter that allows the loss to be proportional to the pressure in the pipe section [28].

Once the failure rate (R) is obtained as a function of the PGV (which depends on Mw

and r), the expected failures are defined as a function of the length of the network. These faults are randomly located through the vector x assuming a uniform distribution.

2.3. Performance Metrics Towards Risk

Performance metrics are used to define numerically the behavior of a lifeline under specific conditions. Network metrics are widely used to interpret their behavior, but they vary according to the infrastructure. Ostfelds (2004) has categorized the performances for water distribution system into three groups: (1) connectivity/topological, (2) hydraulic, and (3) entropy as a reliability surrogate [29].

Connectivity or topological metrics only require the topological information of the network such as nodes (junctions, tanks, and reservoir) and edges (pipes and valves). Despite lacking operation parameters given by the hydraulic analysis, the application of graph theory allows to calculate intrinsic parameters of the network as redundancy or critical routes.

The hydraulic performances correspond to those that emerge from the simulation of a water distribution network hydraulic analysis. These metrics work with the output parameters of the hydraulic analysis, such as pressure, node demand, and flow. Although the objective is the same for all the metrics, the parameters used for the calculation are varied

The entropy performances use the information entropy to estimate critical path or most vulnerable edges. The concept of entropy was first introduced by Shannon (1948) and modified by different authors to generate a most useful expression in their specific fields [30].

Table 2.1: Performance metrics

<table>
<tr><td rowspan="5">Hydraulic</td><td>Reliability based on pressure</td></tr>
<tr><td>Reliability based on satisfied demand</td></tr>
<tr><td>Vulnerability</td></tr>
<tr><td>Sustainability</td></tr>
<tr><td>Resilience</td></tr>
<tr><td rowspan="2">Connectivity/ topological</td><td>Meshedness</td></tr>
<tr><td>Betweenness centrality</td></tr>
<tr><td rowspan="2">Entropy</td><td>Entropic degree</td></tr>
<tr><td>Modified entropic degree</td></tr>
</table>

2.3.1. Reliability, Vulnerability and Resilience

In a hazard scenario, it is necessary to assess what percentage of the critical infrastructure under analysis remains in the operating zone (reliability) and how far away from optimal performance the system will be working (vulnerability). If the system's capacity to recover the optimal state (resilience) is also known, a complete projection of the system's performance during and after the hazard event will be available.

The concepts of reliability and vulnerability are especially important when examining the ability of critical infrastructure to provide continuity in operation. Broadly defined, reliability refers to the probability that a given element in a critical infrastructure system is functional during its lifetime. That is, reliability is a probabilistic measure of elements in a critical infrastructure system and their ability to not fail or malfunction, given a series of established benchmarks or performance guidelines [31].

In contrast to reliability, vulnerability is a more wide-ranging concept, with much broader implications. While reliability focuses on the possibility of maintaining the performance of critical infrastructure elements, vulnerability focuses on the potential for disrupting these elements or degrading them to a point where performance is diminished [31].

As mentioned above, resilience is the capability of overcoming stress or failure conditions [17]. Knowing the resilience in a lifeline allows us to know if there is sufficient capability in the system to allow to overcome local failures and to guarantee the optimal performance to users.

2.3.2. Reliability based on pressure

Reliability based on pressures corresponds to a reliability measure in which a value is assigned to each node according to its operational pressure. The operational states are defined as poor performance, increasing performance, operational performance and decreasing performance [32]. These states are defined by minimum, maximum and operational pressures and shown in Fig. 2.2. The minimum pressure is defined as the necessary in case of fire [33].

Table 2.2: Pressure performance threshold

Pressure performance (in m)	
Pmin	14
Pmax	63
Operational range	28-58

The reliability of the water network is defined by the sum of the operational state indexes of each node multiplied by the weight of each node.

$$R_i = \sum_{j=1}^{N} w_{j,i} I_{i,j} \tag{2.9}$$

$$w_{j,i} = \frac{D_{j,i}}{\sum_{j=1}^{N} D_{j,i}} \tag{2.10}$$

Where $R_{i,j}$ is the reliability of the network for the i^{th} scenario, $I_{j,i}$ is the performance of the j^{th} node for the i^{th} scenario, and $D_{j,i}$ is the demand of the j^{th} node for the i^{th} scenario.

2.3.3. Reliability based on demand

The reliability based on demand is an index that indicates how many of the totality of the nodes do not have deficit. Network reliability is obtained as the percentage of satisfied nodes and total nodes [34].

$$R_i = \frac{\text{No. of times } De_{j,i} = 0}{N} \tag{2.11}$$

Where N is the number of nodes and $De_{i,j}$ is the deficit of the j^{th} node for the i^{th} scenario. D_e is the difference between the node demand and the delivered flow if the delivered flow is minor than the node demand and 0 otherwise.

2.3.4. Vulnerability

Vulnerability based on deficit index, represents the average failure, and it is defined as the total deficit over the total demand [35]. The equation that models this is presented below.

$$V_i = \frac{\sum_{j=1}^{N} De_{j,i}}{\sum_{j=1}^{N} D_{j,i}} \tag{2.12}$$

Where $V_{i,j}$ is the Vulnerability of the network for the i^{th} scenario, $D_{j,i}$ is the demand of the j^{th} node for the i^{th} scenario, and $De_{j,i}$ is the deficit of the j^{th} node for the i^{th} scenario.

2.3.5. Resilience

The concept of resilience in a water distribution system corresponds to the ability to satisfy the demand prior to a damage event. In these terms, resilience relates to the energy the system relies on to restructure water delivery [17].

The following equations in this section were developed by Todini (2000) and are rewritten to generate greater understanding of the methodology.

In order to define the resilience index, several concepts must be defined beforehand. First, the total available power at the entrance is denoted as:

$$P_{tot} = \gamma \sum_{k=1}^{N_r} Q_k H_k \tag{2.13}$$

Where γ is the specific weight of water, N_r is the number of reservoirs, Q_k and H_k are the discharge and the head pressure in the h^{th} reservoir, respectively. The following relationship exist:

$$P_{tot} = P_{int} + P_{ext} \tag{2.14}$$

Where P_{int} is the power dissipated by friction and $P_{ext} = \gamma \sum_{i=1}^{N_n} q_i h_i$ is the power delivered to the users. Where N_n is the number of nodes, q_i and h_i are the delivered flow and head pressure in the i^{th} node, respectively.

Resilience index is defined as:

$$I_r = 1 - \frac{P^*_{int}}{P^*_{max}} \tag{2.15}$$

Where $(\cdot)^*$ are associated to the threshold of satisfied demand and pressure, $P^*_{int} = P_{tot} - \gamma \sum_{i=1}^{N_n} q^*_i h_i$ is the power dissipated in the network to satisfy the total demand and $P^*_{max} = P_{tot} - \gamma \sum_{i=1}^{N_n} q^*_i h^*_i$ the maximum power that would be dissipated internally in order to satisfy the constraints in terms of demand and operational pressure head at the nodes.

Replacing the terms previously written in Eq. 2.15, resilience index is defined as:

$$I_r = \frac{\sum_{i=1}^{N_n} q^*_i (h_i - h^*_i)}{\sum_{k=1}^{N_r} Q_k H_k - \sum_{i=1}^{N_n} q^*_i h^*_i} \tag{2.16}$$

2.3.6. Sustainability

Sustainability index is defined by Loucks in order to unify the parameters, such as reliability, vulnerability and resilience [36]. Thus, sustainability index is defined as the geometric mean of all performance metrics considered.

$$SI_i = \left[\prod_{j=1}^{M} C_{m,i} \right]^{\frac{1}{M}} \tag{2.17}$$

Where SI_i is the sustainability index for the i^{th} scenario, M is the number of metrics that are considered in the equation and $C_{m,i}$ is the m^{th} metric for the i^{th} scenario.

2.3.7. Meshedness

Meshedness coefficient correspond to the ratio of actually present loops to the ideal case of maximum number of loops and it is used as a redundancy measure to capture the status of loops in the network [37]. Settlements that are more meshed have more efficient path systems at the geometric level [38]. More efficient paths between nodes, allowing greater connectivity between the network and less energy loss due to flow displacement.

Meshedness coefficient is defined as the fraction between the total and the maximum number of independent loops in a planar graph.

$$M = \frac{m - n + 1}{2n - 5} \tag{2.18}$$

Where m is the number of edges and n the number of nodes.

2.3.8. Betweenness centrality

Betweenness centrality was introduced by Freeman (1977) as an indicator of the relevance of nodes in network interconnectivity. Then, in order to integrate to this analysis the importance of the location of the nodes corresponding to the reservoirs. $N = n/m$ auxiliary nodes are added to each reservoir, where n and m are the number total number of nodes and reservoirs respectively [39].

Given a node i, and two nodes s and t. There are a number of topological shortest paths between s and t defined as m and a fraction of these pass through node i. The sum of those fractions considering all the tuples s and t is the betweenness centrality of node i [40]. In other words, betweenness centrality is the summation of the fraction of times that node i is on the shortest path between two nodes in the network. The greater the value of betweenness centrality, the greater the relevance of the node in network connectivity. Betweenness centrality is defined as follows.

$$B_i = \sum_{s \neq t \neq i \in V} \frac{\sigma_{s,t}(i)}{\sigma_{s,t}} \tag{2.19}$$

Where B_i is the betweenness centrality of node i, $\sigma_{s,t}(i)$ is the number of shortest paths from node s to node t passing through the node i, $\sigma_{s,t}$ is the number of all shortest paths from node s to node t, and V is the set of nodes belonging to the network

2.3.9. Entropic degree

Entropic degree allows a measurement of the importance of nodes considering weights on edges. For this, it takes into a count three main factors:(i)the strength of connection in terms of the weights of the edges; (ii) the number of edges connected with the vertex; and (iii) the distribution of weights among the edges [41].

$$g_i = \left(1 - \sum_j p_{i,j} \log p_{i,j}\right) \sum_j w_{i,j} \tag{2.20}$$

$$p_{i,j} = \frac{w_{i,j}}{\sum_j w_{i,j}} \tag{2.21}$$

where $w_{i,j}$ is the weight of the edge that connect nodes i and j, $p_{i,j}$ is the normalized weight of the edge between nodes i and j. These parameters are modified in terms of analyzing a water distribution network. In this way, the following equations are obtained.

$$p_{i,j} = \frac{q_{i,j}}{\sum_j q_{i,j}} \tag{2.22}$$

Where $q_{i,j}$ is the flow in the edge between nodes i and j.

2.3.10. Modified entropic degree

Flow entropy consists pipe flow entropy, and nodal demand entropy. This is insensitive to pipe diameter variation, but the diameter value has significant impact on reliability. Therefore, dimensionless parameter $\frac{C}{V_{ij}}$ is introduced to weight the flow entropy and a novel definition of flow entropy termed as diameter-sensitive flow entropy (DSFE) is proposed [42].

$$S_i = \left(\frac{Q_i}{T_i} \log \frac{Q_i}{T_i} + \sum_j \frac{C}{v_{i,j}} \frac{q_{i,j}}{T_i} \log \frac{q_{i,j}}{T_i} \right) T_i \tag{2.23}$$

$$T_i = \sum_j q_{i,j} + Q_i \tag{2.24}$$

Where $q_{i,j}$ is the flow in the edge between nodes i and j, Q_i is the demand of the i^{th} node, vi, j is the velocity in the edge between nodes i and j, and C is an arbitrary velocity constant, e.g., 1 m/s.

In order to redefine the node entropy as an entropic degree coefficient, the Eq. 2.23 is modified as follows.

$$g_i = \left(1 - \frac{Q_i}{T_i} \log \frac{Q_i}{T_i} - \sum_j \frac{C}{v_{i,j}} \frac{q_{i,j}}{T_i} \log \frac{q_{i,j}}{T_i} \right) T_i \tag{2.25}$$

2.4. Behaviour of the water network for known failure rates

The performance metric H depends on the characteristics of the network $\boldsymbol{\theta}$, the failure rate R and the location of each damage $\mathbf{x}$, such that $H(\boldsymbol{\theta}, R, \mathbf{x})$. For a known failure rate, the behavior of the network could be described as:

$$P(H > H_{thresh} | \boldsymbol{\theta}, R) = \int I_F(\boldsymbol{\theta}, R, \mathbf{x}) p(\mathbf{x}) d\mathbf{x} \tag{2.26}$$

where $I_F(\boldsymbol{\theta}, R, \mathbf{x})$ is an indicator function that takes the value of 1 if $H(\boldsymbol{\theta}, R, \mathbf{x}) > H_{thresh}$ and 0 otherwise. Then, P corresponds to the probability of the performance to exceede H_{thresh} given $\boldsymbol{\theta}$ and R.

For the case in which the failure rate R is not directly known, but the seismic characteristics are known (such as magnitude M_w or rupture radius r). Eq 2.26 can be modified using the methodology presented in section 2.2. Where it is defined that the failure rate R can be described as a function of the PGV. In addition, PGV is defined by the moment magnitude M_w and the rupture radius r through the ground motion prediction equation. In this way, the behavior of the network can be described as:

$$P(H > H_{thresh} | \boldsymbol{\theta}, R(M_w, r)) = \int I_F(\boldsymbol{\theta}, R(M_w, r), \mathbf{x}) p(\mathbf{x}) d\mathbf{x} \tag{2.27}$$

Finally, if only the probability distribution associated with the seismic events is known, such as the Gutenberg-Richter parameters for the magnitude distribution or a local tectonic study for the probability of the rupture radius. The behavior of the network for the performance metric H can be described as:

$$P(H > H_{thresh} | \boldsymbol{\theta}, R(M_w, r)) = \int \int \int I_F(\boldsymbol{\theta}, R(M_w, r), \mathbf{x}) p(\mathbf{x}) p(M_w) p(r) d\mathbf{x} dM_w dr \tag{2.28}$$

Chapter 3

Introducing Aging in the Risk Assessment Framework

The characteristics of the internal structure of the pipe vary over time. Although the most significant variations are defined by the diameter and roughness, research have shown that the loss of carrying capacity was due much more to the increase in roughness than to the decrease in pipe diameter [43].Different research have focused on determinate the increase of pipe roughness with the age of pipe. In the following subsections, the models proposed by various authors for the modeling of temporary deterioration will be presented.

3.1. Pipe roughness as a function of time and initial roughness

Mielcarzewicz and Pelka demonstrated a correlation between roughness and initial roughness of the pipe with respect to time [44]. Correlation was obtained on the analysis of steel and cast iron pipes with diameters from 100 mm to 400 mm [14]. The equation developed is presented below.

$$r(t) = r_i + 0.104t \tag{3.1}$$

where $r(t)$ is roughness at time t, and r_i is the initial roughness.

By the other hand, Koppel propose a different equation, based on the calibration of a water distribution network. This network had more than 77 km of pipes. These were made of steel and cast iron and ranged in age from 0 to 41 years [15].

$$r(t) = 15.3 - (15.3 - r_i)\left(\frac{41 - t}{41}\right)^{0.5} \tag{3.2}$$

where $r(t)$ is roughness at time t, and r_i is the initial roughness.

The last equation presented in this work, corresponds to the research developed by Abdel-Monim. For the development of the correlation, 0.5 m cuts of pipes of different ages are tested. The materials used are steel and cast iron [16]. The following equation can be used for periods less than 50 years.

$$r(t) = r_i \left(C_1 \left(\frac{t}{50} \right)^2 + C_2 \left(\frac{t}{50} \right) + C_3 \right) \tag{3.3}$$

where $r(t)$ is roughness at time t, r_i is the initial roughness and the constants C_i are defined in the Table 3.1.

Table 3.1: Values for C_1, C_2 and C_3 defined by Abdel-Monim

Tipy of pipe	C1	C2	C3
Cast iron	43.602	-3.414	0.100
Steel	168.05	7.564	-1.424

3.2. Life-cycle Performance

In order to represent the temporary deterioration in the metrics, the network is analyzed in different life cycles. Under the Poisson assumption of earthquake occurrence, as considered in the case study. Thus, the probability of exceeding the threshold of a metric over a life cycle of the water distribution network is defined as:

$$P(H > H_{thresh}|\boldsymbol{\theta}, t_{life}) = 1 - \exp^{-t_{life}\nu P(H>H_{thresh}|\boldsymbol{\theta},\text{seismic event})} \tag{3.4}$$

Where t_{life} is the life cycle considered, ν is the expected number of events per year and $P(H > H_{thresh}|\boldsymbol{\theta}, \text{seismic event})$ is the probability of exceeding the metric threshold given that a seismic event has occurred . This probability can be solved by the generic risk integral of Eq. 2.28.

Chapter 4

Computational Considerations

Whereas the network performance presents a probabilistic behavior since the location of the damages are not known. Eq.2.26 could be solved via stochastic simulations, where N samples drawn from $p(\mathbf{x})$ are used to evaluate the network, such that Eq.2.26 could be estimated as:

$$P(H > H_{thresh}|\boldsymbol{\theta}, R) \approx \sum_{i}^{N} \frac{I_F(\boldsymbol{\theta}, R, \mathbf{x_i})}{N} \tag{4.1}$$

However, the computational burden to perform thousands simulation is practically unfeasible for practical purposes. In this regard, it is decided to employ small number of samples (couple of hundreds). Here, two thresholds are selected, the first one corresponding to probability of exceedance equal to 0.5 and another with a probability of exceedance equal to 0.9. The former case corresponds to a risk-neutral case while the later corresponds to a risk-adverse attitude.

In the same way, Eq 2.27 could be solved via stochastic simulations, where N samples drawn from $p(\mathbf{x})$ are used to evaluate the network, such that Eq.2.27 could be estimated as:

$$P(H > H_{thresh}|\boldsymbol{\theta}, R(M_w, r)) \approx \sum_{i}^{N} \frac{I_F(\boldsymbol{\theta}, R(M_w, r), \mathbf{x_i})}{N} \tag{4.2}$$

Finally, Eq. 2.28 also is solved via stochastic simulations, where N,M and L samples drawn from $p(\mathbf{x})$, $p(Mw)$ and $p(r)$ respectively are used to evaluate the network , such that Eq.2.28 could be estimated as:

$$P(H > H_{thresh}|\boldsymbol{\theta}, R(M_{wk}, r_j)) \approx \sum_{k}^{L} \sum_{j}^{M} \sum_{i}^{N} \frac{I_F(\boldsymbol{\theta}, R(M_{wk}, r_j), \mathbf{x_i})}{L \cdot M \cdot N} \tag{4.3}$$

Since the number of hydraulic analyses is defined by the number of repair rates $(L \cdot M)$ multiplied by the number of random damage distributions on the network (N). It is preferred to solve Eq. 4.1 for a defined number of R and apply an interpolation function G. This decreases the computational cost N times for Eq.4.3.

$$G(R^*) = interp(P(H > H_{thresh}|\boldsymbol{\theta}, \mathbf{R}), R^*) \tag{4.4}$$

Where $G(R^*)$ is the result of the interpolation, $\mathbf{R}$ is a vector of R on which the performance was calculated, and R^* the value to interpolate. Finally, the performance of the network for the distribution of probabilities of magnitude and rupture radius is defined as:

$$P(H > H_{thresh}|\boldsymbol{\theta}, R(M_{wk}, r_j)) \approx \sum_{k}^{L} \sum_{j}^{M} \frac{G(R(M_{wk}, r_j))}{L \cdot M} \tag{4.5}$$

Chapter 5

Water Distribution Network Case Study

5.1. Network Characterization

This section presents an example to illustrate the application of the proposed framework. Which illustrates the application of the proposed framework to estimate the time-variant seismic reliability, vulnerability, and resilience of a real water distribution network subject to gradual deterioration due to increase of pipe roughness. The network model corresponds to real data. The network has 2 reservoirs, 8 valves, 2156 nodes, and 2422 pipes. The pipes are made of cast iron and has a total extension of 100.6 km. The minimum pipe diameter is 50mm, the maximum is 600mm, and the median value corresponds to 100mm. The mean characteristics are presented in Figure 5.1.

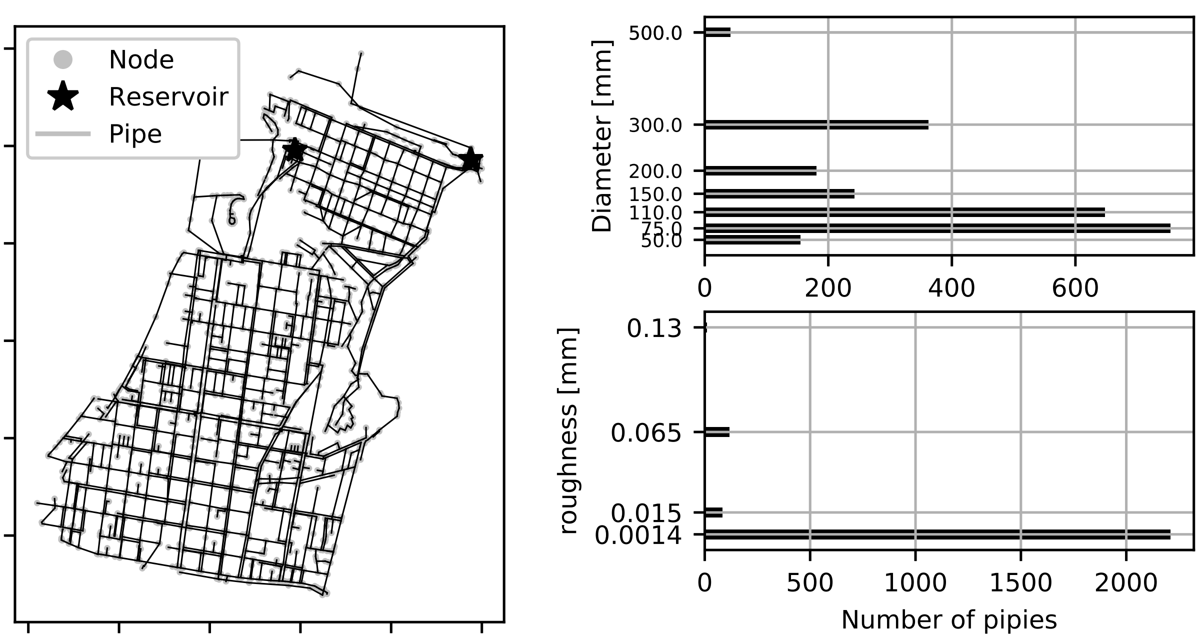

Figure 5.1: Water network characterization: (left) water distribution network model showing nodes, reservoirs and pipes; (up right) pipe diameter distribution; (bottom right) pipe roughness distribution.

5.2. Hydraulic Analysis

Hydraulic Analysis of Water Distribution Network is executed by EPANET2 [45] in Python using library EPANETTOOLS. This software develops a demand-driven analysis, an analysis that assumes that pipelines are always full and pressurized. This is problematic for damaged networks given that the existence of leaks or ruptures create higher demands that, due to the continuity equations, create negative pressures. These analysis tend to overestimate the available flow in the system and provide wrong estimations of the system operability. For this reason, the library by itself could not conduct a damage simulation. To develop a correct analysis is necessary to generate an algorithm in Python to modify the network. The algorithm, applied to treat negative pressures, is based on previous research and consists in modifying the existing network [46][47]. While Markov was the first to generate a methodology for treating negative pressures, Chou [48] modified this methodology and implemented a model that allowed the simulation of seismic damage to water distribution networks. This methodology is used in the illustrative example.

The treatment of negative pressures proposed by Chou can be summarized in the following steps (Fig. 5.2). The first step consists in running the hydraulic analysis to find the minimum pressure. The algorithm is complete if that value is not negative. Otherwise, the next step is to remove the most negative node and adjacent elements such as links or valves. The previous step can generate sub-networks non-connected to any reservoir nor tank. The solution to this problem is track all non-connected nodes using graph techniques and remove all of them and their adjacent elements. The final step is to run the modified network and check if all negative pressure points were removed. After this modification process of the network is possible to get all the hydraulic analysis parameters.

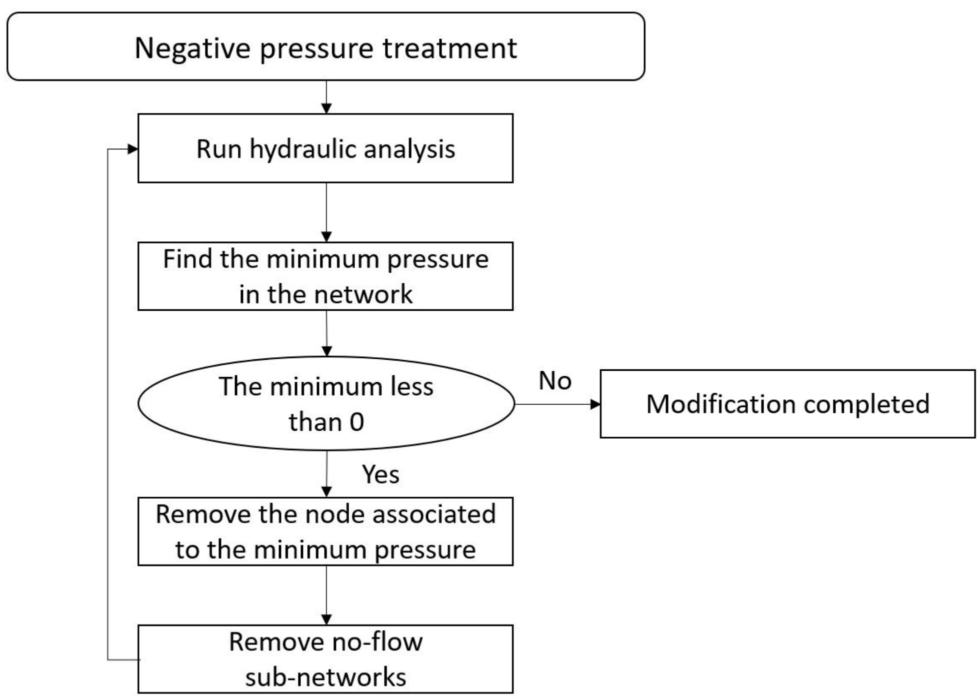

Figure 5.2: Negative pressure treatment diagram

5.3. Creation of Hydraulic Models

The creation of the hydraulic models to be analyzed are made based on the original model. Sub-models are derived from the original model based on the deterioration of its pipes. From the sub-models, hundreds of damaged sub-models are obtained according to a defined failure rate. The negative pressures of the damaged sub-models are treated to obtain the final models.

The final models are a function of the failure rate and the age of the network. These models will be analyzed through the different metrics to obtain correlations of the network's performance in the face of seismic events that occur throughout the time.

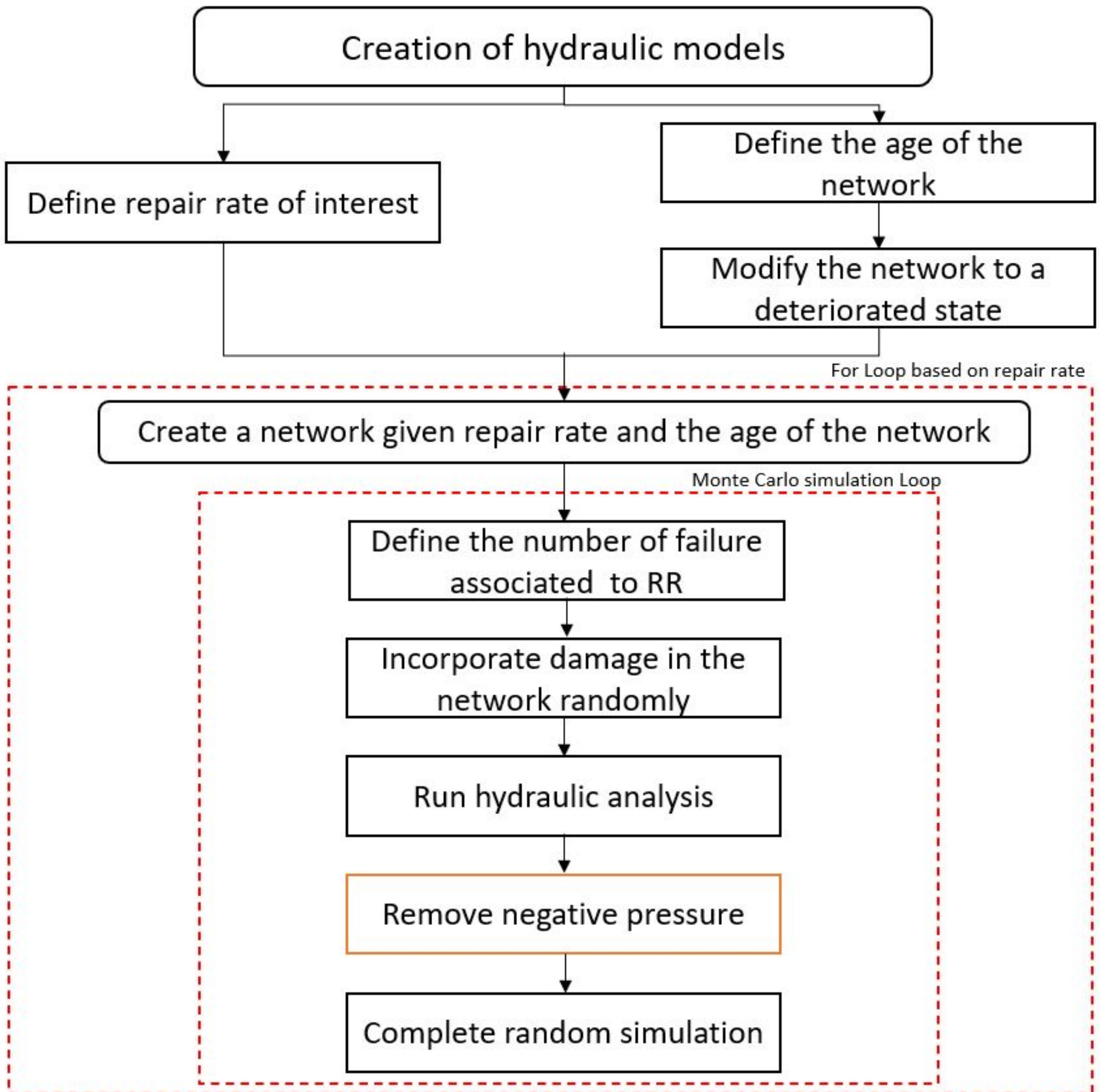

Figure 5.3: Creation of hydraulic models diagram

5.4. Critical path estimation

An estimation of the critical path is made through the calculation of criticality of the nodes. This is done using 3 metrics: entropic degree, modified entropic degree and betweenness centrality. The entropic degree and modified entropic degree require knowledge of the flow in the pipes, while the betweenness centrality only requires knowledge of the network topology.

The calculation of the critical path was calculated without considering temporary or seismic deterioration. This is because the temporary deterioration is applied in an equitable manner on the network without significantly altering the flow or topology. As for the seismic deterioration, entropy increases, but a critical path cannot be defined given the randomness of the failure.

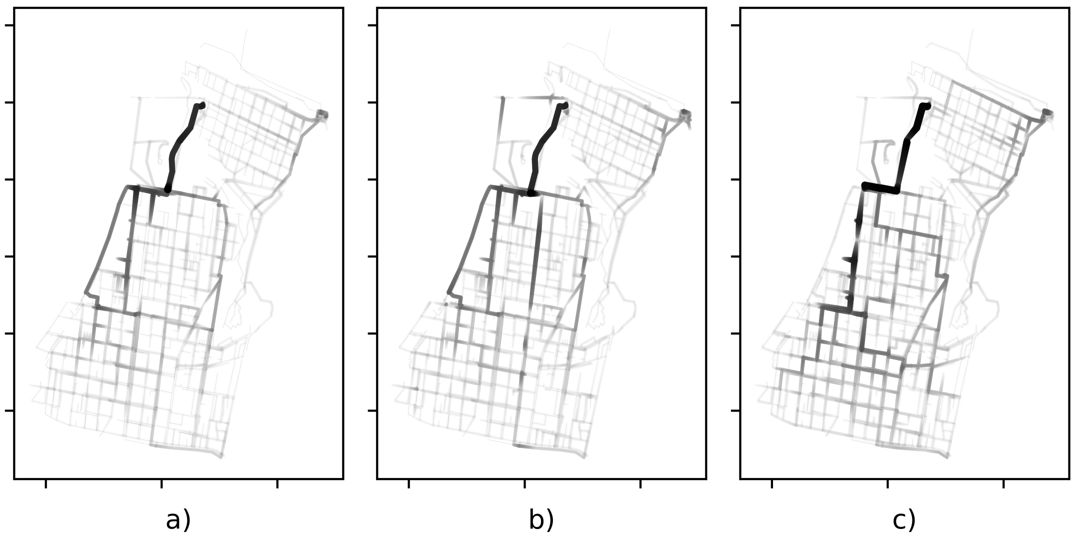

Figure 5.4: The critical route defined by the entropic degree presents three main vertical routes. By considering the diameter as a variable and therefore the flow speed in the calculation of entropic degree.The path defined by modified entropic degree (b) presents four main vertical paths. Betweenness centrality (c) does not consider the physics of the problem (higher flow implies higher energy loss) so it underestimates peripheral paths and overestimates the central paths.

5.5. Network Performance Analysis under Seismic Risk and Ageing Deterioration

For temporary deterioration, the model proposed by Mielcarzewicz and Pelka (1997) is used. Based on the non-deteriorated model, the roughness parameters are modified according to Eq.5.1, where r_i corresponds to the roughness of the i^{th} pipe, r_0 corresponds to the roughness of the i^{th} pipe in time 0, and t corresponds to time.

$$r(t) = r_0 + 0.104t \tag{5.1}$$

Once the model is modified to represent the corresponding age. A linear space of repair rate values is generated. The failures associated with the repair rates are integrated with the methodology presented in section 4. The damage integration process is performed hundreds of times to find the neutral and adverse risk state for each metric as defined in section 2.3. The metrics reliability based on pressures, reliability based on satisfying demand, vulnerability, resilience, and sustainability index are calculated using the methodology previously defined in section 4. The results are presented in Figures 5.6 and 5.5. Once the fragility curves associated with the repair rate have been developed, the seismic scenarios are generated assuming a Poisson distribution and independence with previous events. The uncertainty in moment magnitude is modeled by the Guttenberg-Richter relationship truncated on the interval $[M_{min}, M_{max}] = [7, 9]$ (event smaller than M_{min} do not generate significant effect in the network) which leads to $P(M) = b_M e^{-b_M M_{min}}/(e^{-b_M Mmin} - e^{-b_M Mmax})$ and expected number of events per year $\nu = e^{a_M - b_M M_{min}} - e^{a_M - b_M M_{max}}$. The regional seismic factors a_M and b_M are chosen considering the values presented by Poulos (2019) for zone 3 [49]. It is considered a constant hypocentral depth of 60 km. The rupture radius is obtained by following a beta distribution with a minimum rupture radius of 30km, a maximum of 150km and an average of 70km. Since the minimum radius is considerably larger than the distance between the two furthest nodes in the network, it is considered a single distance between the system and the earthquake.

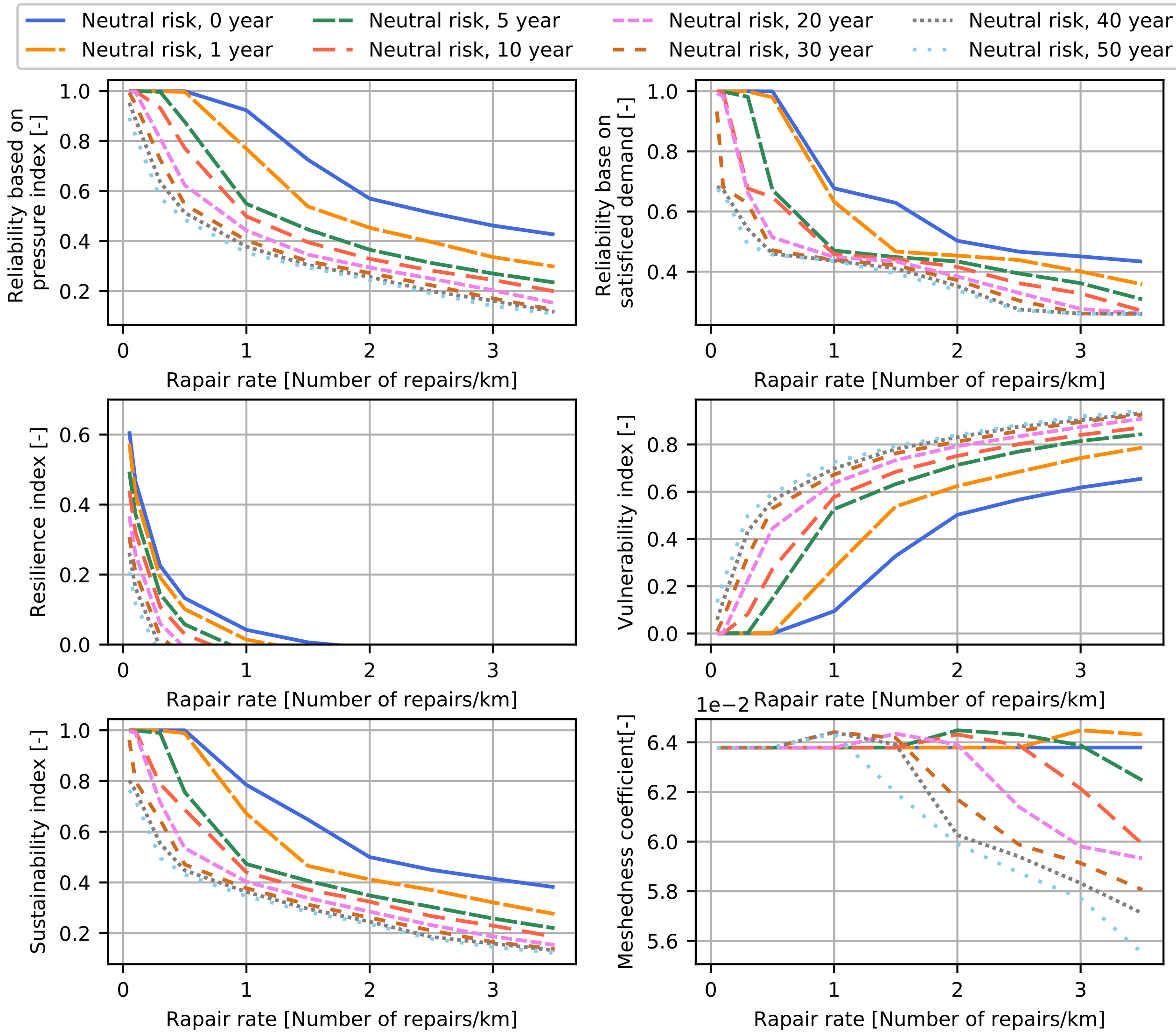

Figure 5.5: Deterioration due to aging of pipes captured by the metrics of (a) Reliability base on pressure, (b) reliability based on satisfied demand, (c) Resilience, (d) vulnerability, (e) sustainability index and (f) meshedness coefficient for neutral risk tends to converge over a period of 50 years.

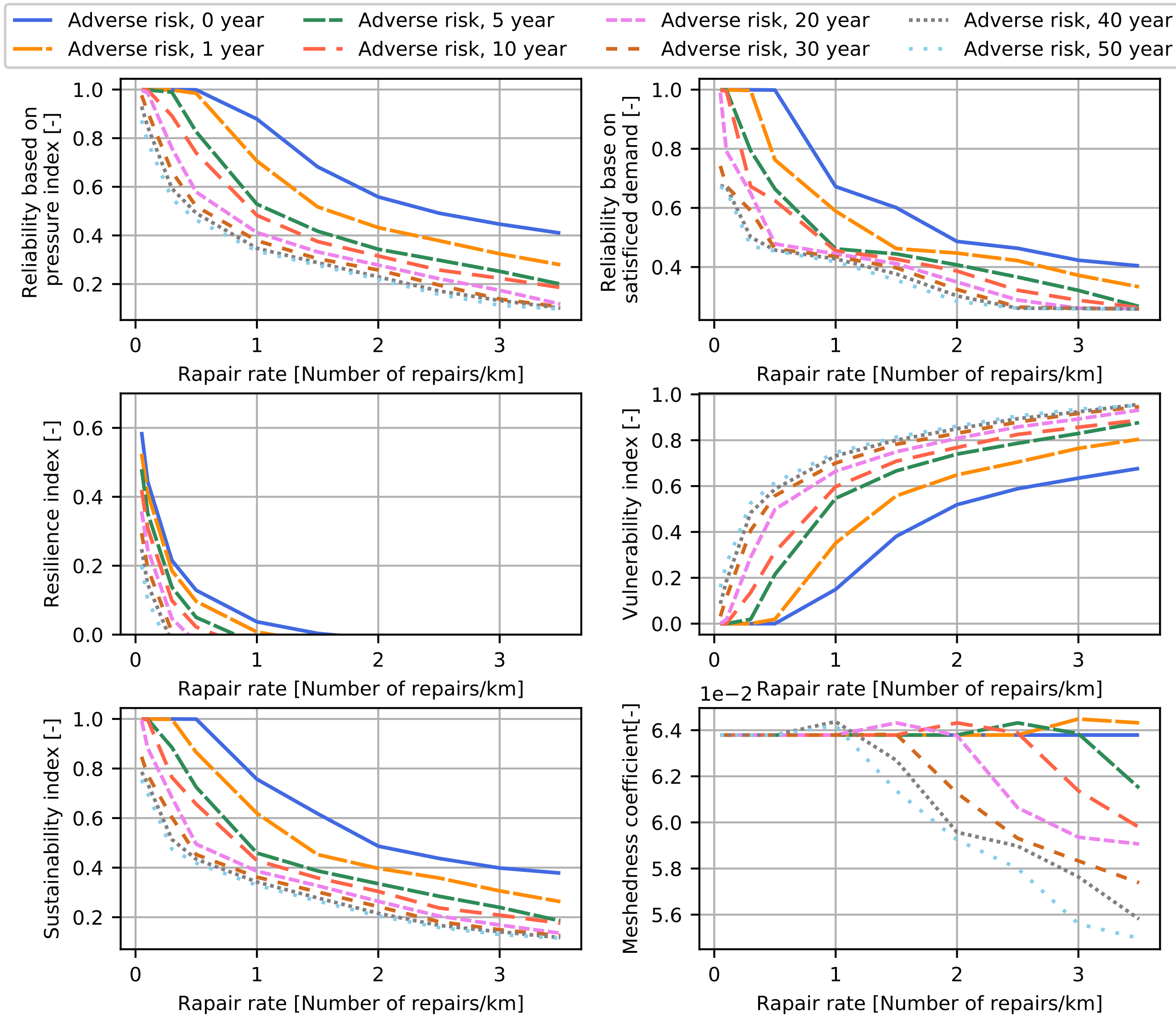

Figure 5.6: (a) Reliability base on pressure, (b) reliability based on satisfied demand, (c) Resilience, (d) vulnerability, (e) sustainability index and (f) meshedness coefficient for adverse risk.

PGV is calculated from the ground motion prediction equation developed by Molas (1995) presented in Eq.5.2 [50]. Meanwhile repair rate is calculated from the relation defined by Milashuk (2012) presented in Eq.5.3. As mentioned above the PGV equation is a function of the moment magnitude and the rupture distance to the nodes. In addition, the distance between the farthest nodes is 2 km. Thus, the average distance to the rupture zone is much larger than the greatest distance between two nodes. This is why an equal rupture radius is considered for all nodes and thus a constant repair rate for the entire network.

$$\log_{10}(PGV) = -1.769 + 0.628 \cdot M_w - \log_{10}(r) - 0.0013 \cdot R + 0.00222 \cdot H \quad (5.2)$$

$$R = e^{1.72 \cdot log(PGV) - 8.67} \quad (5.3)$$

The network metrics for each year studied are obtained by cubic interpolation on the curves presented in Figure 5.7. The values corresponding to the probability of exceeding 50 % and 90 % of the scenarios are selected through the rearrangement of the equations section 4.4. In this way, the curves associated with the probability of exceeding 50 % and 90 % over time are obtained for the neutral and adverse risk scenarios. These results are presented in Figure 5.7.

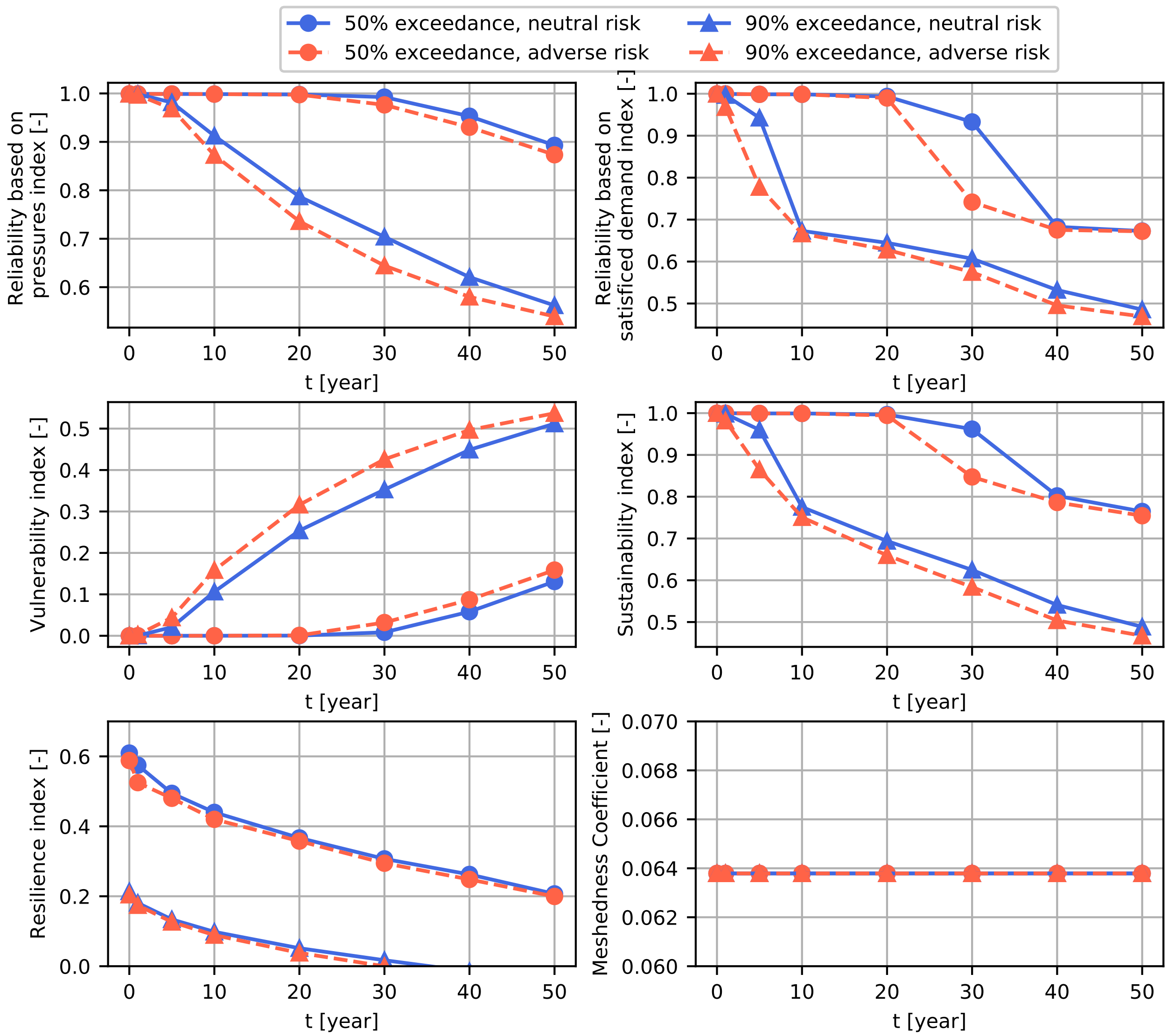

Figure 5.7: The seismic scenario to which the network is submitted (50 % exceedance or 90 % exceedance) has more influence on the expected value of the metric than the behavior of the network defined by the location of its faults (neutral or adverse). The behavior described above is followed by all the metrics: (upper left) Reliability base on pressure, (upper right) reliability based on satisfied demand, (middle left) Resilience, (middle right) vulnerability, (bottom right) sustainability index and (bottom left) meshedness coefficient for neutral and adverse risk, considering aging deterioration.

Chapter 6

Results Analysis and Discussion

The critical path is estimated by entropic (2) and topological (1) methods. All methods define the reservoir pipeline as the most critical. Well-defined routes are observed for entropic methods, which are presumed to have greater connectivity and flow. By considering the diameter as a variable, and thus the velocity of flow, it is possible to visualize an extra vertical route (Fig. 5.4b). The estimation of the route by topological methods (Fig. 5.4c) presents different results in relation to entropic methods. There is an overestimation of the paths located to the center. This overestimation is mainly due to the fact that the topological method only considers connectivity between nodes and does not include physical variables such as distance between nodes, flow in the pipes, diameter of the pipes and frictional losses.

With respect to the computational cost of the calculation of critical routes In the entropic methods no difference is observed, since the maximum cost is given by the hydraulic analysis that delivers the flow in the pipes. The computational cost of the topological method is lower for the network being worked on, because there are functions that deliver the weight of the node and only request the network's graph. For more complex networks the computational cost could increase substantially, since for the calculation of the Betweenness centrality all the possible routes between 2 nodes are studied, and this problem scales in an exponential way.

The information required by the entropic methods is greater than that required by the topological method. While the entropic methods require all the necessary information to perform a hydraulic analysis (connectivity, diameters, roughness coefficient, node demand, etc.), the topological method only requires connectivity between nodes.

The centrality metric Betweenness centrality is a feasible method in the estimation of the critical path when there are few data from the water distribution network. This method allows to define some of the critical paths defined by more complex methods. In case of having enough data to estimate the flow of the pipes, it is recommended to use the modified entropic degree method for the estimation of the critical path.

For a constant and randomly distributed number of failures, there is no significant difference between the average value of the metrics and their 90 % adverse value. This means that it is possible to estimate the performance of the network when the repair rate of the system is known. With the data presented in the Figures 5.6 and 5.5, a seismic analysis is performed using the relationship between PGA and repair rate. This methodology allows to reduce the computational calculation $M \cdot L$ times, where M and L are equal to the amount of seismic events used.

The reliability metrics present a similar behavior, being the reliability based on pressures a smoother curve. This is because the reliability based on pressures is the weighting of an index by node that goes from 0 to 1, while the reliability based on satisfied demand is a binary index 0 or 1. vulnerability index, which is calculated as the deficit on demand, also has a logarithmic behavior. The sustainability index is the geometric average of the reliability index based on deficit and the additive inverse of vulnerability. This index reaches lower values than the reliability index due to the nature of the geometric average.

The resilience index is the only index that reaches 0. A value of 0, given the index formulation, indicates that for equal or greater repair rates the system will not be able to restructure to satisfy the initial demand.

The mesh coefficient has erratic values along the repair rate. The coefficient should not increase its value when the system is damaged. Its value increases because, with the elimination of damaged nodes and disconnected pipes, the network is modified increasing its value. These variations make it an imprecise coefficient, in addition, it only varies its initial value once there are negative pressures or great deterioration in the network.

With respect to the estimation of the value of the metrics in time 5.7, it is obtained that the seismic scenario to which the network is subjected (50 % excess or 90 % excess) has greater influence on the expected value of the metrics than the location of the faults (neutral or adverse). The form followed by the metrics corresponds to the seen in Figures 5.6 and 5.5. It can be seen that the mesh coefficient does not provide any information.

Chapter 7

Conclusions

This thesis proposes a general framework to model the impact of temporary deterioration on the seismic risk assessment of water distribution networks. Conducting a performance evaluation through various metrics. This article also allows to observe the behavior of the metrics on a real topology.

The flexibility of this methodology allows it to be modified to the specific requirements of each research or project and its local characteristics (such as ground motion prediction equations, repair rate and PGA relationships, Gutenberg-Richter law parameters, etc.).

The obtaining of network performance metrics based on a uniform sampling of failure rates allows to reduce the number of processes needed to generate an accurate measurement of adverse scenarios. In addition, this methodology allows the hydraulic analysis process (associated with the higher computational cost) to be performed only in the generation of the performance metrics curves.

In general, expected results are obtained for the metrics. This indicates that they can be used to measure the performance of a water network. Only the meshedness coefficient has an erratic behavior. This behavior is because the coefficient indicates the connectivity of the network. The connectivity only varies when modifying the topology, which is not necessarily related to a worse performance.

Three methods are evaluated to define the critical paths. The modified entropic degree allows for consideration of flow velocity resulting in more routes than the classical entropic degree. Betweenness centrality is not consistent with the other two metrics, although it could be used as a first approach, since it only requires knowledge of the network topology. The metrics associated with the entropic degree require knowledge of the flow in the pipes and its directionality, which is derived from the hydraulic analysis.

Since all metrics are defined by the state of the node which is then weighted to obtain the performance of the network. It is possible to visualize which zones have worse performance and thus generate mitigation measures to improve network performance.

The development of this work evidences the impact of aging at the moment of evaluating the performance of a water network affected by a seismic event. And the importance of generating maintenance policies for the water network system, considering a constant replacement and monitoring of its pipes.

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